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CENTRAL BOARD OF IRRIGATION INDIA

सत्यमेव जयते

GOVERNMENT OF INDIA

ANNUAL REPORT

(TECHNICAL)

OF THE

CENTRAL BOARD OF IRRIGATION INDIA

1948

PART II

EDITED BY

N. D. GULHATI, I. S. E., M. I. E. (INDIA)

SECRETARY

सत्यमेव जयते

GOVERNMENT OF INDIA
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JANUARY, 1949.

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FOREWORD

This Annual Report contains an account of research and other technical investigations on subjects connected with irrigation and hydro-electric, carried out in India, Burma and Ceylon during the year 1947. It is published both for the information of irrigation and hydel engineers in India, Burma and Ceylon and to make known in other countries the problems of irrigation and hydro-electric development in this part of the world and how they are being over-come.

Summary reports for the work carried out during the year 1947 at the nine research stations, and papers on specific subjects selected by the Board, together with the discussions thereon, appear under the respective subject heads. The discussions constitute the technical proceedings of

(*i*) The Eighteenth Annual Meeting of the Research Committee, August, 1948.

(*ii*) The Nineteenth Annual Meeting of the Central Board of Irrigation, December 1948.

in addition to which an informal meeting of the Research was held in April, 1948.

The Preliminary Notes were written by the Secretary for the information of those attending the Research Committee and Board Meetings.

It is regretted that due to lack of space, it has not been possible to include in full all articles contributed to the Research Committee Meeting. Abstracts or abridgments of articles not included in full are, however, provided. In other cases where abstracts or explanatory introductions were not supplied by authors, the Secretary has provided introductory abstracts.

The undersigned would repeat the request, contained in previous reports, for the supply of information relating to any of the problems mentioned in the publication. He will gladly furnish further details of any of the experiments, investigations, or other information relating thereto.

SIMLA;
Dated, January, 1949.

N. D. GULHATI,
Secretary,
Central Board of Irrigation.

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SECTION C--HYDRAULIC WORKS

IC. Limitation of Model Experiments and Comparison of Models and Prototype

PRELIMINARY NOTE

This subject has been discussed by the Board since 1939 under the title 'Field results of measures adopted after model experiments' with the object of comparing the results obtained from works built or remodelled in accordance with indications obtained from model experiments in order to assess the value and limitations of the use of models in solving problems connected with irrigation and river training works. It is particularly desired to make a study of such cases where the results on the prototype are different from those on the models with a view to developing the technique of experimentation. At the 1945 annual meeting, the Board passed a resolution that a Board Publication on the construction and operation of Hydraulic models be issued, based on the contributions by research officers. In response to this resolution, Central Water ways, Irrigation and Navigation Research Station; Engineering Research Laboratories, Hyderabad; Irrigation Research Division, Poondi; and Irrigation Research Division, Sind have sent in their notes. The Director, Irrigation Research, Punjab, has sent in an advance copy of the contribution which had yet to be approved by the Chief Engineer (East Punjab). Bombay, United Provinces and Ceylon have no contribution to make. The Director, Irrigation Research Institute, West Bengal, has promised to contribute a note on the subject which is still awaited. Mysore has not sent any reply so far.

The note on 'The Development of electrical analogy methods in India' promised by Dr. V. I. Vaidhianathan is still awaited.

It is suggested that it will also give a better indication of the advantages of the model experiments if a short account of the savings in the cost of works effected by model experiments is also reported. For this purpose it is proposed that the title of this subject head be changed to 'Model Experiments' and divided into following sub-heads:

(*i*) Limitations of model experiments and comparison of models and prototype.

(*ii*) Economy effected by model experiments.

At the last informal winter meeting of the Research Committee a reference was made regarding the use of plastic for the preparation of models. Shri S. L. Malhotra, Director, Irrigation Research, East Punjab has since written to say that he has found a mention of transparent plastic in the book 'Hydraulic Model Studies for the design of Bhakara Dam—Hydraulic Laboratory Report No. Hyd. 210'. According to him this book has been received in the East Punjab Irrigation Department in connection with the design of Bhakara Dam from U.S.A.

A letter has been sent to the Director, U.S. Waterways Experiment Station, Vicksburg, Mississippi (U.S.A.) by the Secretary enquiring details about transparent plastic materials which are being used for the preparation of models.

The following items were discussed at the 1947 Research Committee meeting :

(1) Model limitations as regards erosion and accretion upstream and downstream of tree groynes.

(2) Model experiments in connection with scour hole below head regulator over Rohri Canal.

Recent Literature.

(1) Model studies of spillway and bucket for Centre Hill Dam, Caney Fort River Tennessee. War Department, Corps of Engineers, U.S. Waterways Experiment Station, Technical Memo No. 202—1, Vicksburg, Mississippi.

(2) Model study of spillway End Dam, Yocona River, Mississippi. War Department, Corps of Engineers, U.S. Waterways Experiment Station, Technical Memo No. 2-223, Vicksburg, Mississippi, March 1947.

(3) Use of models in design of spillways. Commonwealth Engineer, Vol. 34, No. 12, July 1, 1947.

(4) Allen J.—Scale models in hydrulic engineering. Longman, Green and Co., London, New York Toranto, 1947.

(5) Model study of spillway and silting basin Harlan County Dam, Republican River, Nebraska—U.S. Waterways Experiment Station, Vicksburg Mississippi, Technical Memo No. 2-236, September 1947.

(6) Chatley, Herbert, D. Sc. (Engineering), M.I.C.E. Consulting Hydraulic Engineer, Bath.—The Distortion of scales in models with Looze Beds—Second meeting of the International Association for Hydraulic Structures Research, Stockholm, 1948, Paper No. 1.

THE YEAR'S WORK

The following items were discussed at the 1948 Research Committee Meeting :

(1) Protecting the right bank of the Luni River at Balotra by means of a repelling spur.

(2) Scour below the Central sluices of the Mahanadi anicut at Cuttack (Orissa).

(1) PROTECTING THE RIGHT BANK OF THE LUNI RIVER AT BALOTRA BY MEANS OF A REPELLING SPUR [1]

In consequence of the abnormal rainfall in the Aravali Ranges, the main line of the Jodhpur Railway between Parlu and Gole was severely damaged. The river spilled over the banks between Balotra and Tilwara causing extensive breaches. In addition to other recommendations for the protection of the Railway embankment [2], to prevent attack at Balotra a *repelling* spur was recommended in 1944. Since then, the floods in the river were very low and hence the spur has not been tested under high flood conditions, but even with continued low floods the spur, according to the local officers, is working satisfactorily. The main channel of the river, which—even when the rainfall was normal—used to run along the side of the spur, has been repelled by the spur and now flows at a distance of 300 feet from it.

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(2) SCOUR BELOW THE CENTRAL SLUICES OF THE MAHANADI ANICUT AT CUTTACK—ORISSA [1]

Some modifications were suggested in the design of the Central Sluices of the Mahanadi Anicut. A comparison of the scour observed before and after the modification, showed that, while under the old conditions, the deep scour-hole was almost abutting the pavement with imminent danger of undermining and collapse of the pavement, after the modifications there was a silting at this point to the extent of 20 feet.

The deepest bed contour, 263 feet away from the dwarf wall No. 4, was previously at R. L. 15, while after the modifications it had accreted by 20 feet or more.

Further, at the left corner between walls No. 3 and 4, the apron had collapsed leaving deep scour-hole, but after the modifications at this point the bed has silted by more than five feet.

It was clear that the conditions had markedly improved since the construction of the energy dissipating device recommended by the station.

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(1) Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report Technical 1947, page 34.

(2) Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report Technical, 1945, item No. 12.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. S. T. GHOTANKAR introducing item (1) said that for the protection of the Railway embankment, a repelling spur was recommended in 1944. Since then, the floods in the river were very low and the spur had not been tested under high flood conditions ; but even under continued low floods the spur, in the opinion of the local officers, had served its purpose satisfactorily. The main channel of the river which used to run alongside of the spur, had been repelled by the spur and now flowed at a distance of 300 feet from it.

Introducing item (2) he said that a comparison of the scour patterns below the Central Sluices of the Mahanadi Anicut, before and after the modifications suggested had been carried out, revealed that the deep scour-hole had silted to the extent of 20 feet.

DR. H. L. UPPAL complained that the reports were received by him very late.

Regarding the limitation of model experiments, he said one particular model could answer the problem for which it was constructed. It might have a lot of limitations, if during the investigations some other problems on that verp subject were received for investigation. For instance, if there were a river model under investigation they adopted a certain scale. When they had a river in which a weir was to be constructed, then the same scale and model would not be suitable for solving all the problems. Exaggeration was very essential in a river model, but it was quite difficult in river studies when there was a structure like a weir in it. Therefore, in view of this, the river models should not be distorted. He had a discussion with Dr. Mullard on this. They did not distort the model at all. They kept it for solving all the problems that arose on the subject.

MR. D. V. JOGLEKAR thought that undistorted models could not give any useful results.

MR. S. T. GHOTANKAR said that the limitations pointed out by Dr. Uppal had already been reported by the Poona Station two years back.

DR. N. K. BOSE enquired about the scale to which models were reduced in the Punjab.

DR. H. L. UPPAL said that at Malikpur they had very few geometrically similar models. In unrestricted models vertical exaggeration had to be given. The scale used by them was 1/100 and 1/120 and models were made of very fine material. They made a very big model in which they did not give distortion. They only distorted the slope in those river models which were restricted.

DR. N. K. BOSE said that he knew, in Germany they had been using fine material, which was usually light. But unfortunately, they could not get that in India in large quantity. As they were to work with ordinary sand which they had got accustomed to, he thought they should stick to that. They

had yet to find the importance of the difference in the various problems they were going to investigate. Use of distortion was restricted to particular problems only. For example for standing wave investigations no distortion was necessary.

DR. H. L. UPPAL said that only a limited number of problems or a particular problem for which a model was constructed could be investigated with that model. Neither the river conditions upstream of the particular reach of observation nor downstream could be ascertained by the model. It could only be effective for investigation for the particular reach for which the model was made.

THE CHAIRMAN (S. MAN SINGH) said that a reference was made to the Director U. S. Experiment Station regarding the use of plastics in models. A reply had been received which was read out as below:—

" Plastics have been used for a number of years in this country for constructing, models of various hydraulic structures such as conduits, intake towers, turbine scroll cases and the like where visual observations of flow conditions are desired. Typical examples of such use are illustrated by the enclosed photographs.*

Although a large variety of plastics are available commercially, two of those found best suited for model construction are sold under the trade-names of " Lucite " and " Plexiglas ". Detailed information on these products may be obtained by contacting the following :

(a) E. I. Dupont de Nemours and Co. Inc., Plastics Dept., 626 Schuyler. Ave., Arlington, New Jersey. This concern manufactures " Lucite " and publishes a pamphlet concerning its use entiled " The Lucite Manual ".

(b) Rohm and Haas Co., 222 West Washington Square, Philadelphia 5 Pennsylvania are producers of " Plexiglas " and publishers of " The Plexiglas Design Manual ", " Plexiglas Fabricating Manual " and " Plexiglas—Crystal—Clear Plastic."

Both of the above mentioned plastics are readily formed (after heating), cut, drilled, machined or cemented. They may be obtained in sheets of various colour, size and thickness (1/16" to 2") and in the form of rods up to a diameter of 2 inches. " Lucite" may also be obtained in tubes up to 6 inches in diameter. Prices, are practically the same for either product.

MR. N. S. GOVINDA RAO said that these plastic models got scratches due to the sand that they used and after some time they would not be able to see anything through them.

DR. H. L. UPPAL said that plastic glass was better and did not scratch by the action of sand.

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*Recorded in the office of the Central Board of Irrigation.

THE CHAIRMAN (S. MAN SINGH) said that in the informal winter meeting some research officers had said that they had had difficulties in obtaining certain equipments and apparatus from abroad on account of exchange difficulties *etc.* The Government of India was approached and they had asked that actual instances of delay should be referred to them. He then drew attention to the proposals made by the Secretary regarding introduction of a new sub-head on " Economy effected by model experiments ".

RAO BAHADUR D. V. JOGLEKAR said that there were two kinds of savings direct and indirect. For instance, if a barrage was designed after model experiments, the actual saving in construction costs might not be large but the indirect benefits would be many. On the other hand if the cost of a project was estimated at a certain figure and by carrying out model experiments, they were able to save something on construction, maintenance and other charges, that was a direct benefit. It was always very difficult to calculate the indirect advantages.

It was agreed that the proposal be accepted and the subject head should be as follows :

1C. Model Experiments.

(i) Limitations of model experiments and comparison of models and prototype.

(ii) Economy effected by model experiments, direct and indirect.

With reference to Dr. Uppal's complaint regarding late receipt of annual report, it was decided that in future each research station should send to every other research station two copies of its annual report instead of one as in the past.

The Secretary said that Dr. Vaidhianathan had promised to write a note on the work that had been carried out in India relating to " Electrical anology experiments ".

He requested Dr. Vaidhianathan to state when he expected to complete the work.

DR. V. I. VAIDHIANATHAN replied that he was in Delhi and had none of the records with him ; he had left some 800 diagrams in the West Punjab. He would have to come to Simla and see old records in the Central Board of Irrigation Library, but they had given only a few diagrams in their Annual Reports. These would not be enough and he had nothing else with him.

THE SECRETARY remarked that they had done quite a bit of work in India and it would be unfortunate to lose it.

Dr. V. I. Vaidhianathan said that it would not be difficult to repeat the work, in fact with the experience gained, the results would be more correct.

Sardar Bahadur Sarup Singh said that Mr. S. L. Malhotra would give all assistance.

Dr. V. I. Vaidhianathan said that U. P. could also give him assistance.

Rai Bahadur M. C. Bijawat suggested that since the Central Drilling School, Roorkee was close to the U. P. Research Station it would be useful to get some work done there.

It was decided that Dr. Vaidhianathan would divide the work between East Punjab and U. P. and have his note ready by July 1949.

DISCUSSION BY THE BOARD

The Secretary said that two items were discussed at the Research Committee meeting. The Reseach Committee had proposed that this subject be renamed as below.

10. Model Experiments

(*i*) Limitations of model experiments and comparison of models and prototype.

(*ii*) Economy effected by model experiments, direct and indirect.

As regards the proposed Central Board of Irrigation Publication on models the position was the same as stated in para 1 of the Preliminary Note on page 597.

Continuing The Secretary said that as decided by the Board, in 1945, Dr. Vaidhianathan was to write a draft for the proposed Central Board of Irrigation Publication on 'Development of Electrical Analogy methods in India'. At the Research Committee meeting Dr. Vaidhianathan stated that all his records had been left in West Punjab and he could prepare a draft only if he was given the facilities of repeating his experiments at some Research Stations in India. The decision of the Research Committee was given above. Intimation had since been received from the Chief Engineers U. P. and East Punjab that they would provide all facilities to Dr. Vaidhianathan to repeat his experiments in their respective research organisations.

At the last informal winter meeting of the Research Committee, a reference was made regarding the use of plastics for the preparation of models and Secretary was asked to make enquiries in this respect. Since then the Secretary had been in correspodence with E. I. Du pont de Nemours and Company, U. S. A. and had been able to obtain literature on the subject. A price list of Lucite materials had since been sent to all research stations. A chain of 32 samples lucite sheet colours received from the Company was passed round for inspection.

RAI BAHADUR S. K. GUHA enquired about the thickness of plastics.

DR. H. L. UPPAL said that they were obtained in sheets varying from $\frac{1}{4}$ to $\frac{3}{4}$ inch. The cost of a sheet 4 feet $\times$ 3 feet with $\frac{1}{4}$ inch thickness was about rupees 250 to rupees 300.

Speaking generally about model experiments DR. UPPAL said that one model would not suffice. For an investigation it might be necessary to have at least three models instead of one.

DR. N. K. BOSE expressed the difficulty he was experiencing in reproducing waves in models. They had considerable wave action which he was not able to reproduce properly. He enquired of R. B. Joglekar how he was managing to simulate waves of required range in a model.

SHRI D. V. JOGLEKAR said that waves of the required range could be obtained with a wave producing machine. Such a machine was being evolved at the Poona Station.

DR. N. K. BOSE asked as to how waves were measured.

SHRI D. V. JOGLEKAR said that they had not come to any final conclusion in this matter. He said it was possible to have a wave recording apparatus ; such an apparatus was being set up at the Poona Station.

RAI BAHADUR C. L. HANDA said that the subject of model experiments was generally viewed by the research worker and by the engineer in slightly different perspective. He just wanted to put up to the Board the idea Dr. Bose had voiced. In order to complete a small investigation on a model if the engineer thought that just one model would be ample, or that one model should give all the details that were required, that opinion was likely to be falsified and the engineer should be prepared to the necessity of having a number of models as proposed by the research worker. He found that the research worker was apt to consider that each model was giving different kinds of results, one of them giving positive results and the other some sort of negative results. In respect of the positive experiment there might be justification to assume that something had been repeated. But he wanted to warn the engineer that he had to be very very careful and conservative in blindly accepting negative evidences given by the model. He thought that it was rather hazardous to assume the conditions, if they did not appear on the model. What he wanted to stress was that the number of models required to study the various aspects of the same problem might be accepted by the engineers, as capable of sub-divisions of a special model. Hydraulics, he said, could be evolved in conflict with the large number of years

experience of the engineers and if any result on the model confirmed that experience, well and good. If on the other hand something happened showing the negative evidence he wished to give a note of caution to the engineers in accepting them.

Rao Bahadur D. V. Joglekar too spoke on the usefulness of having a large number of models *e.g.* in Sukkur Barrage experiments they had to have five different models. He agreed with Shri Handa about the caution that had to be observed regarding negative results. Especially with regard to silting and scouring one had to be very careful in interpreting. There were definite serious limitations in models regarding depth of scour *etc.*

Dr. N. K. Bose agreed with Mr. Joglekar regarding silting. But by looking at the model and as well as the prototype one could safely predict whether the silting or scouring were likely to occur. But it could not be expressed quantitatively. But another point about which Mr. Handa spoke, cut at the root of all model work. He wanted models to be repeated. If one could interpret the results properly and by judicious use of the models in different ways, he could get at the results without repetition.

Rai Bahadur R. R. Handa endorsed what Rao Bahadur D. V. Joglekar had said. He spoke about the river diversion works he carried out in Orissa and how the results from models were helpful.

THE BOARD APPROVED THE PROPOSAL OF THE RESEARCH COMMITTEE REGARDING THE CHANGE IN THE NAME OF THE SUBJECT.

2C. River Control

PRELIMINARY NOTE

This subject first appeared on the agenda of the Research Committee, in 1942, under the head " River behaviour, training and control ", though the subject " Meandering of rivers " had been under discussion for many years past. In 1947 ' River behaviour ' was separated with its two sub-heads : (*i*) Floods and (*ii*) Stage-discharge relation, and was included under Section A-Hydrology, item 2. The subject " River Control " is to be discussed under six sub-heads :

(*i*) Flood protection works, (*ii*) River training works, (*iii*) Scour and erosion, (*iv*) Spill, (*v*) Meandering, (*vi*) Tidal action including that in estuaries.

(i) Flood protection works

This sub-head appeared on the agenda for the first time in 1945. Certain aspects of this subject were discussed previously in different forms ; in 1939-40 as " Collection and study of statistical data pertaining to flood control ".

The fol'owing items were discussed at the 1947 Research Committee meeting :

(1) Redistribution of sand load between the Mahanadi and Katchur systems at Cuttack—Orissa

(2) Flood absorption in Krishnarajasagar

(3) An unprecedented flood in Mula and Pravara valleys and estimate of loss of flood damage

(4) Meandering of rivers and flood control—by Shri K. B. Ray

Recent Literature.

(1) Report of the Mississippi River Commission—United States, Government Printing Office, Washington, 1947.

(2) Multipurpose development of Indian rivers—Science and Culture, Vol. 13, No. 1, July 1947.

(3) Voorduin W. L.—Multipurpose development of the Tennessee River—Science and Culture, Vol. 12, No. 11, May, 1947.

(4) Lawson, L. M.—International treaty authorizes flood control structures on Lower Rio Grande—Civil Engineering, Vol. 17, No. 3, March 1947.

(5) Protecting highways from damage by floods—Public Works, Vol. 77, No. 2, February 1946.

(6) Fox, J. M.—First venture flood project—Western Construction News Vol II, No. 8, August 1946.

(7) Fall river dam –Construction Methods, Vol. 29, No. 7, July 1947.

THE YEAR'S WORK

The following items were discussed at the 1948 Research Committee Meeting :—

(1) Bituminous reinforced hotmix revetment for river bank protection.

(2) Redistribution of sand load between the Mahanadi and the Katjuri systems at Cuttack (Orissa).

(1) BITUMINOUS REINFORCED HOTMIX REVETMENT FOR RIVER BANK PROTECTION (1)

ABSTRACT

Flood and breaches of embankments by the River Damodar is quite frequent and large sum of money and great loss of property and crops are incurred every time such breaches occur. The irrigation and Waterways Department spend a large sum of money each year in bank revetment work strengthening those places which appear likely to give trouble during the next flood and maintaining and repairing works carried out in previous year. This note describes the experiment carried out on the preparation of bituminous reinforced hotmix canal lining.

Two general methods of bank protection are followed : a flexible brick mattressing and bamboo or salballah revetment driven into the bank at threatened places.

The brick mattressing consists of three or four layers of brick laid sideways and lengthways alternately wrapped in 4 inch mesh 10 or 12 gauge wire-netting, the whole being suspended from heavy stakes firmly embedded in the top of the bank. As erosion takes place at the bottom of the bank the brick-mattressing follows the scour by dropping down in the form of a flap which effectively prevents further erosion by sealing the partially disintegrated bank.

The second method of employing bamboos or salballahs is a more rigid type of construction and is not so effective under a powerful current but is considerably cheaper.

The cost of brick mattressing is very high. So the Irrigation Department would welcome cheaper forms of construction which would be flexible enough to answer the purpose.

Consequent on Baron Von Asbeck's recent visit to Calcutta who met the Chief Engineer, Irrigation and Waterways, for discussion on the use of bitumen for hydraulic works which was carried out in the other parts of the world, the Chief Engineer was sufficiently impressed and agreed to carry out a trial locally, with specific regard to bank protection, in co-operation with Messrs. Burmah Shell & Co.

He was, however, unwilling to try out an unknown material on the bank of the river itself as the consequence of failure at that point would be too serious and therefore suggested a site on the Damodar canal just below one of the canal regulator gates where considerable erosion of the bank takes place every year. A length of approximately 130 feet of the canal was selected at Galsi in the district of Burdwan in West Bengal where the trial was to be carried out.

(1) River Research Institute, West Bengal, Annual Report, 1947, pages 60-62.

LABORATORY INVESTIGATION METHOD

In connection with the above work for the river bank protection trial at Galsi, the Institute was requested by Messrs. Burmah Shell and Co. to undertake experiments on the preparation of bituminous reinforced hotmix canal lining for river bank protection.

This type of revetment consists of two inches thick reinforced bituminous hotmix of fine asphaltic concrete or sheet asphalt type laid on a properly graded and compacted slope of natural soil.

The composition of the hotmix should be such as to make it an impervious and stable bituminous layer after compaction, so that it can prevent erosion of soil on which it was compacted.

The idea of the bituminous mix is to reduce the voids in the final mix to minimum. In case of hydraulic works a mix with 2 or 3 per cent. of voids in the mix will be regarded as water tight under the atmosphere pressure.

Particles retained on 10 mesh (A.S.T.M.) sieve is termed as coarse aggregate. Particles passing 10 and retaining on 40, passing 40 and retaining on 80, passing 80 and retaining on 200, all combined are termed as fine aggregate. Particles passing 200 are termed as filler. Here cement is used as filler. For stabilisation the aggregate comprising the mix should be properly graded. As it is not possible to get a single mixing for the whole requirement, there cannot be a particular proportion of grading in the mix. So at the different sites of work every blending is individually graded. A somewhat typical composition according to Baron Von Asbeck is given in Table 2 C.1.

TABLE 2 C.1

	Fine asphaltic concrete per cent	Sheet asphalt Per cent
¼″—10 mesh	25	..
10—40 mesh	30	38
40—80 mesh	20	30
80—200 mesh	10	17
Filler (cement)	15	15
	100	100
Bitumen	9	10

Bitumen in the mix serves the purpose of the binder which makes the void as minimum. Care should be taken that excess of bitumen is not used. Otherwise mix will be soft and will begin to flow on slope at high temperature. A rough checking of the amount of bitumen used is to close a sample of the mix in a brown paper and lightly tapped. The stain left on the paper is carefully observed. It should neither be too dark nor too light. The stability of the bitumen can be increased to a certain extent by adding more filler. Bitumen to be added entirely depends on the void of the compacted dry blended aggregate. It is calculated like this.

Real and apparent densities of the dry aggregate are determined from which the pereentge of the void is calculated.

Percentage of voids in volume—

$$= \frac{\text{Real Density} - \text{Apparent Density}}{\text{Real Density}} \times 100$$

$$= \frac{\text{Real specific gravity} - \text{Apparent specific gravity}}{\text{Real specific gravity}} \times 100$$

The voids thus found in the dry aggregate should have to be filled up by bitumen.

Percentage of bitumen by weight to 100 parts of the dry aggregate $= \frac{\text{Percentage of voids by volume in the aggregate}}{\text{Apparent specific gravity of the aggregate}} \times$ specific gravity of bitumen $\times 100$.

Specific gravity of the bitumen is taken as 1·04.

A sample of the mix thus prepared is subjected to the determination of real and apparent densities from which the percentage of the voids in the mix is determined. As stated above, if the percentage of voids in the mix is found to be 2 or 3, the mix is watertight for the purpose.

It is now tested for its imperviousness to water and stability against flow on slope at high temperature. Imperviousness is tested by subjecting the sample to a head of 200 feet of water for 24 hours. There should be no flow of water through the sample. Stability against flow on slope is tested by placing a block of 2 in. $\times$ 7 in. $\times$ $3\frac{5}{8}$ in. of the sample of the mix on a slope of 1: 1 at 170° F and its displacement is to be noted. There will be a slight displacement at first after which there should be no more movement. If the rate of flow is higher it can be checked by using a harder grade bitumen which suits the climate or adding more filler. But care should be taken not to exceed a certain limit beyond which the voids start increasing. In some cases crushed stone chips may be used.

Once the mix has been properly prepared, works can be started. But the precaution to be taken of uniformly compacted subgrade, laying of uniform thickness, proper recking, *etc.* But the most essential is the proper compaction of the mix with heavy roller.

After the preliminary investigation in the laboratory field work was carried out on the bed and side of Damodar canal downstream of the fall at Galsi. The work has been completed and the results are being watched with interest. A complete report will be submitted in next year's annual report.

(2) REDISTRIBUTION OF SAND LOAD BETWEEN THE MAHANADI AND THE KATJURI SYSTEMS AT CUTTACK - ORISSA [2]

The Katjuri River takes off from the Mahanadi just below the Naraj gorge five miles upstream of Cuttack. At peak flood 56% (8·4 *lakhs* cusecs) discharge flows down the Katjuri at Naraj, whereas the rest of the discharge flows down the Mahanadi. In addition to the two spills, namely Birnasi Spill (Q=70,000 cusecs at peak flood) on the left bank and Banki Road Spill (Q= 1,00,000 cusecs at peak flood) on the right bank, the Katjuri bifurcates into Kuakhai, Surua and Katjuri loop in the neighbourhood of Cuttack.

The fundamental source of trouble has been the excess sand charge entering the Katjuri river at Naraj[3]. A series of experiments done with a view to improving the distribution of sand load between the Mahanadi and Katjuri river have been already reported [4].

In continuation of those experiments the following experiments were carried out :—

Series II

Experiment No. 2 : (Repeated) With Banki Road spill discharge at peak flood reduced to 33,000 cusecs, Birnasi gap fully open and Naraj waterway reduced by 16·85%.

Experiment No. 4: (Repeated) With Banki Road spill discharge at peak flood reduced to 33,000 cusecs, Birnasi gap fully closed by embankment ending in a spur and Naraj weir waterway reduced by 28·5%.

Experiment No. 5 : With Banki Road spill discharge reduced to 50,000 cusecs at peak flood, Birnasi gap closed by embankment ending in a spur but provided with high level escapes at Birnasi spill channel crossings ; and Neraj weir waterway reduced by 20 per cent.

Table 2 C. 2 shows the ratios of sand drawn by the Mahanadi and its branches at Cuttack, the time average discharge, relatives and charge *etc.*

For ready reference results of Series I, Experiment No. 1 with existing conditions and of Series II, Experiment No. 1 which gave optimum results in the series of experiments done last year are also reproduced.

The Experiments No. 2 and 4 of series II which were repeated this year were continued for four more rising and falling flood cycles. It was observed that in both these experiments, the relative sand mix in Mahanadi become richer, and on the Katjuri system, though the Katjuri loop and the Surua were better off, yet the Kuakhai was not. It will be seen that, except in Experiments No. 1

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(2) Central Waterways, Irrigation and Navigation, Research Station, Poona Annual Report, Technical, 1947, pages 35-37.

(3) Central Waterways, Irrigation and Navigation Research Station Poona, Annual Report, Technical 1945, Item 9.

(4) Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report, Technical, 1946, pages 44-52.

TABLE 2 C.2

Showing the ratios of sand drawn by the Mahanadi and branches and the time average discharge.

Details of experiment	Total quantity of sand collected in all branches	MAHANADI			KATJURI LOOP			SURUA			KUAKHAI			Remarks
		$\sum\frac{QT}{T}$	Qs	X	$\sum\frac{QT}{T}$	Qs	X	$\sum\frac{QT}{T}$	Qs	X	$\sum\frac{QT}{T}$	Qs	X	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Series I														
Experiment 1 : with existing conditions.	14.000	1.390	6.000	0.820	0.769	3.000	0.740	1.526	4.000	0.498	0.516	1.000	0.370	(a) Figures denote quantity of sand and the ratios observed in the model.
Series II														
Experiment 1 : Birnasi Gap fully open ; Banki Road spill at peak flood reduced to 50,000 cusecs; Naraj Weir Waterway reduced by 15·5%	(a) 15.000	1.5000	7.000	0.880	0.750	3.000	0.760	1.420	4.000	0.535	0.450	1.000	0.423	(b) Figures denote quantity of sand and ratios after correction to make sand collected=sand injected.
	(b) 14.000		6.550	0.825		2. 80	0.710		3.720	0.499		0.934	0.394	
Experiment 2 (Repeated): with Birnasi Gap fully open ; Banki Road spill Q at peak reduced to 33,000 cusecs ; Naraj weir Waterway reduced by 16·85%	(a) 16.000	1. 58	8.000	0.962	0.770	3.250	0.802	1.320	3.500	0.504	0.490	1.250	0.485	T=Time in minutes X=Charge in parts per 1000 by weight $\sum\frac{QT}{T}$=Ratio of Qs to $\sum\frac{QT}{T}$ where $\sum\frac{QT}{T}$ is the
	(b) 14.000		7.000	0·842		2.850	0.702		3.000	0.440		1.09	0.424	
Experiment 4 (Repeated) : with Birnasi gap closed ; Banki Road spill at peak flood reduced to 33,000 cusecs ; and Naraj weir waterway reduced by 28·5%	(a) 16.250	1. 60	8.50	1. 01	0.720	3. 25	0.858	1.410	3.250	0.438	0.44	1. 25	0.540	weighted mean discharge with respect to time computed by taking the sum of the products of each stage discharge and its time of duration divided by the total time of the flood cycle.
	(b) 14.000		7.32	0.869		2.80	0.738		2.83	0.376		1. 08	0·465	
Experiment 5: with Birnasi embankment ending in spur and provided with high level escapes ; Banki Road spill Q reduced to 50,000 cusecs at peak flood and Naraj weir Waterway by reduced 20%	(a) 16.045	1. 55	7. 75	0. 95	0.730	3. 25	0.815	1.43	3. 92	0.521	0. 46	1.125	0.465	
	(b) 14.000		6. 77	0.830		2. 84	0·737		3. 42	0.455		0.986	0.405	

and 5 of Series II, the relative sand charge of the Mahanadi was slightly worse than under existing conditions. Both the Katjuri loop and the Surua improved but the Kuakhai branch drew 6·45% more sand in Experiment No. 1 and 9·83 % more in experiment No. 5 and still more in Experiments No. 2 and 4.

The results of experiments in series II—*i.e.* with Banki spill Q reduced to 50,000 cusecs, Birnasi gap embankment having high level openings and Naraj waterway reduced by 20%—gave the desired results for all the branches except the Kuakhai and the difference to Kuakhai would be small.

A short embankment without obstructing the Birnasi gap channels, would possibly give the optimum solution, provided the Banki spill discharge is reduced to 50,000 cusecs and Naraj weir waterway reduced by 15·5 % ; because it would, without materially vitiating results of Experiment No. 1, Series II, give a water front to Cuttack for which the embankment is primarily intended.

DISCUSSION BY THE RESEARCH COMMITTEE

Dr. N. K. Bose in introducing item (1) said that these experiments were done in co-operation with Messrs. Burmah Shell and Co., on the downstream side of a fall of Damodar canal near Galsi. It stood fairly well for one season. Dr. Bose could not say anything about the exact cost of this work. His idea was that it was rather heavy.

Introducing item 2, Mr. S. T. Ghotankar stated that the experiments were done during the year in continuation of last year's experiments to improve the distribution of sand load between the Mahanadi and Katjuri River and were being continued.

Mr. Govinda Rao enquired whether any expansion joint had been provided in the revetment for river bank protection item (1).

DISCUSSION BY THE BOARD

The Secretary said that two items were discussed at the Research Committee meeting (page 606). There was no resolution.

It was decided to retain the subject on the Agenda.

(ii) River Training Works

PRELIMINARY NOTE

" Training ". was the original subject head, but it was later divided into,

(*a*) Falling Aprons, (*b*) The heads of guide banks, spurs *etc.*, (*c*) The results of measures adopted in training rivers.

In 1942, the Board passed the following resolution :

"Resolved further that Mr. Inglis be requested to write a note on Meandering, with a data already collected by him and that the Secretary to collect data from the Provinces and the Railways on 'Scour' and 'Training' and pass them on to Mr. Inglis for analysis."

Accordingly a form was issued for the collection of data of launched aprons which data it was intended to analyse and compare with the assumptions made by Springs (5) and observations made on models. Most of the data available have been received, except that from Sind. The data were passed on to the Punjab Irrigation Research Institute, Lahore, for examination and presenting it to the Research Committee.

As regards the collection of the data for the proposed Board Publication on 'River Training', the United Provinces, Bengal, Madras. Bombay, Baluchistan and Sind have sent in their contributions. The Railway Board has also supplied a few notes on the subject.

The Central Waterways, Irrigation and Navigation Research Station has suggested that the Publication of Sir Claude Inglis's book on the subject may be awaited, which would contain the contribution of that station.

Central Provinces and Berar, Orissa, Punjab, Ceylon, Bahawalpur, Mysore and Hyderabad have nothing to contribute. Note promised by Bihar and Burma are still awaited.

The following items were discussed at the 1947 Research Committee meeting :—

(1) Model experiments for the prevention of erosion at Kurigram by the river Dharla.

(2) Erosion of the river Torsa.

(3) Model investigations for the proposed Mor barrage at Tilpara.

(4) Training of the Luni River at Railway Bridge No. 3 of Samdari Raniwara Branch of the Jodhpur Railway—Experiments in connection with the—

(5) Training of the Yamuna River at Delhi Gate Power House by means of a pitched embankment.

(6) Training of Watrak at Kaira, Bombay.

(7) Training of the Sutlej River at Samasata about six miles downstream of the Empress Bridge at Adamwahan N. W. Railway.

(8) The Rupnarain river upstream of the goods yard, Kolaghat Railway Station (upstream of the Bengal Nagpur Railway Bridge)—measures to reduce risk of attack on the right bank of

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(5) Spring, F. : "River training and control in guide-bank system". Railway Board Technical Paper No. 153, Simla, 1903.

(9) Training of Rivers with permeable tree groynes.

(10) River models—Beas river—Railway Bridge at Dhilvan in District Jullundur.

(11) Ravi River—Spur at Baherian, District Gurdaspur.

(12) River Chenab and Jhelum at Trimmu headworks.

(13) River Sutlej at Samasata railway station.

(14) River experiments—Shape of spur aprons for conventional types of spurs.

(15) Effect of the angle of approach on the shape of a T-headed spur.

Recent Literature.

(1) Ronalds A. F.—Harnessing Australia's greatest river—the work of the River Murray Commission. River Murray Commission, Victoria, 1946.

(2) Multi-purpose development cf Indian rivers. Science and Culture, Vol. 13, No. 1, July 1947.

(3) Voorduin W. L.—The multipurpose development of the Tennessee River. Science and Culture, Vol. 12, No. 11, May 1947.

(4) Mirza Mohammed Din—Behaviour of rivers and their training. Central Board of Irrigation Journal, Vol. 4, No. 4, October 1947.

THE YEAR'S WORK

The following items were discussed at the 1948 Research Committee meeting:—

(1) Protection of Kankhal town against floods in the Ganga at Hardwar.

(2) To train the river Ganga upstream of Bhimgoda weir for equalising the flow on weir crest and preventing retrogression downstream.

(3) Model investigations for the proposed Mor Barrage.

(4) Protecting the right bank of the Kuakhai river above the Bengal Nagpur Railway Bridge near Cuttack (Orissa).

(5) Protection of the left bank of Katjuri river at Khannaga upstream of the Katjuri Railway Bridge (Orissa).

(6) Training of the Ganga river at Kanpur.

(7) Kshipra river at Ujjain—Gwalior State.

(8) Measures for arresting bank erosion near Bhadeli and Bhagda villages on the Auranga river.

(9) Protective measures for several railway bridges on Madras—Bombay line of the Madras and Southern Maratha Railway between miles 82 and 113.

(10) Training of the Tapti river at Surat (Bombay).

(11) Musapet Anicut System.

(1) PROTECTION OF KANKHAL TOWN AGAINST FLOODS IN THE GANGA AT HARDWAR (6)

ABSTRACT

Gives details of experiments carried out for the protection of Kankhal Town which is threatened by floods ever since the siting of canal head works.

Kankhal town is situated at the western most stream of the Ganges below Hardwar, which, before the construction of the canal about a hundred years ago, used to get water for spilling from the main stream in floods. In winter, channels were dug for bringing the water for domestic and religious purposes both for Kankhal and Hardwar. With the siting of the canal head-works, the western channel above Hardwar was gradually developed and this started bringing more and more supplies towards Hardwar and Kankhal during the floods. The record high flood of 1894 (510,000) did considerable damage to Kankhal Ghats *etc.* Since then the town of Kankhal has always been threatened by floods inspite of various measures adopted from time to time to train the river. Consequently model experiments to solve this problem were taken in hand.

To start with, the survey of 1942 was laid out with shingle of size 1 inch to 2 inches with horizontal scale $\frac{1}{200}$ and, vertical scale $\frac{1}{50}$. The 1932 survey was not complete as compared to 1939 survey and the entrance conditions of the model were reproduced from the latter. These scales gave no bed movement even with the discharge scale of $\frac{1}{14,000}$ based on $Q_r = L_r^2$, up to a flood of three *lakhs*. At site the bed movement is fairly general at a discharge of one *lakh*.

This material was, therefore, removed and Ranipur sand (average diameter 0·025 in.) mixed with 5% to 10% of shingle up to $\frac{1}{4}$ in. size was used with the scale same as above. This gave good correspondence of gauges upto floods of 60,000 only and thereafter the bed movement was so excessive that the model was difficult to operate. Even by changing the discharge scale to $\frac{1}{20,000}$ based on $Q_r = L_r . D_r^{1.5}$, the bed movement was unmanageable and the gauges became very low.

With this experience in hand of the above extreme cases, it was clear that a suitable mix of bed material had to be evolved which should represent movement of sand at 30,000 cusecs, small shingle upto 3 in. at 50,000, bigger shingle upto 4 in. and smaller boulders at one *lakh* and general bed movement at 125,000. The vertical exaggeration of 1 : 4 was also considered to be excessive for this size of model and consequently the vertical exaggeration was altered from 1 : 4 to 1 : 3. With this vertical exaggeration a mix of about 50% of $\frac{1}{2}$ in. to 1 in. shingle with 25% of $\frac{1}{4}$ in. and 25% of Ranipur sand was ultimately found to give most satisfactory results with a discharge scale obtained by trial of $\frac{1}{22,000}$.

With the gauges satisfactorily reproduced for 1942 onwards, with the above scale, a time scale of $\frac{1}{16}$ was decided. The silt charge was also controlled being

(6) United Provinces Irrigation Research Station Report, on Research Progress during 1947, pages 47—53.

finer material at low floods and mostly heavier material at higher floods in varying quantities increasing with the increase in bed movement. Floods were now run for 1942, 1943, 1944 and 1945 to check the survey after the floods of 1945 *vide* Figure 2C. 1. The correspondence obtained at the model was very encouraging.

Thus the model proved itself quite satisfactory with the following scales which were adopted finally :—

Length scale	$L_r=\frac{1}{120}$
Depth scale	$D_r=\frac{1}{40}$
Time scale	$T_r=\frac{1}{95}$
Discharge scale	$Q_r=\frac{1}{22,000}$

By this time the survey of 1947 was also almost ready and the 1945 survey was further checked by running flood hydrographs of 1946 and 1947.

DEVICES TRIED AT THE MODEL

While proving the model it became abundantly clear that another one or two cycles of floods of intensity in the neighbourhood of $2\frac{1}{2}$ *lakhs* are likely to pull the entire river to the Kankhal channel. In fact while inspecting the junction of Kankhal and Bijnor channels, it was found that only a rapid of extraordinary large sized boulders at the mouth of the Kankhal channel is standing in the way of this catastrophe as, below the junction, the level of Kankhal channel is R. L. 908 while that of Bijnor channel is R. L. 912. The latter being at the inner curve is shoaling and thus inducing more and more clear water in the Kankhal channel.

(*i*) It was suggested by Superintending Engineer I to try the result of extending the present Kankhal *bund* by about 2,000 feet. In doing so, the town of Kankhal was maskedup to Patiala House only, but this did not, and could not, prevent the river turning completely into Kankhal channel. Evidently the result of this device was to shift the trouble further down by 2,000 feet. The downstream portion of the town was still in the danger of submergence at high floods as also of intense bank erosion.

(*ii*) The Chief Engineer had also ordered to investigate the effect of joining the abandoned portion of the old Kankhal *bund* as per request of the people of Kankhal. This was done. The 960 feet gap between the existing Kankhal *bund* and the abandoned portion of the old *bund* was closed in a smooth angle from chainages 2,000 feet of the *bund*. This immediately brought the entire length of the old abandoned *bund* into severe action. Besides such an abrupt turning of the river direction resulted in considerable afflux as well. At the downstream of the abandoned portion, the current again swung to the right towards Daksh Temple. The Bijnor Channel did not develop even with the old *bund* supposed to be restored thoroughly as the main current still persisted towards west.

This device was thus a failure and finally too it would cost a great deal to restore 1,000 feet of the abandoned *bund* to proper strength and raise it to the required height, besides closing the gap of 960 feet width of Kankhal channel.

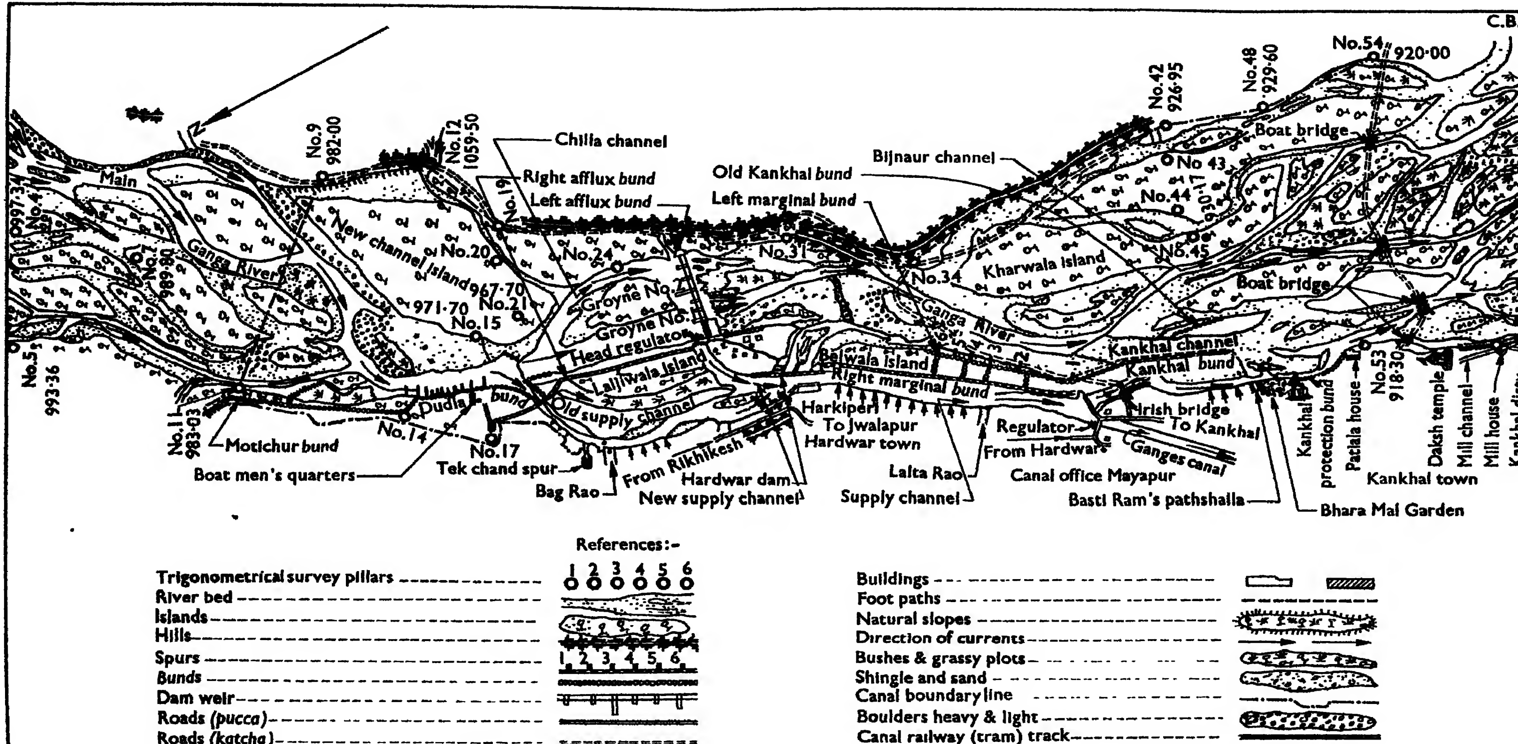


Figure 2C.1 :- *Showing survey plan of Ganga River at Hardwar (after rains of 1945)*

Scale 1 0·5 0 1 2 Miles

Reg. No. 2737 XDD (C) '50-1,210 (P.Z.O.).

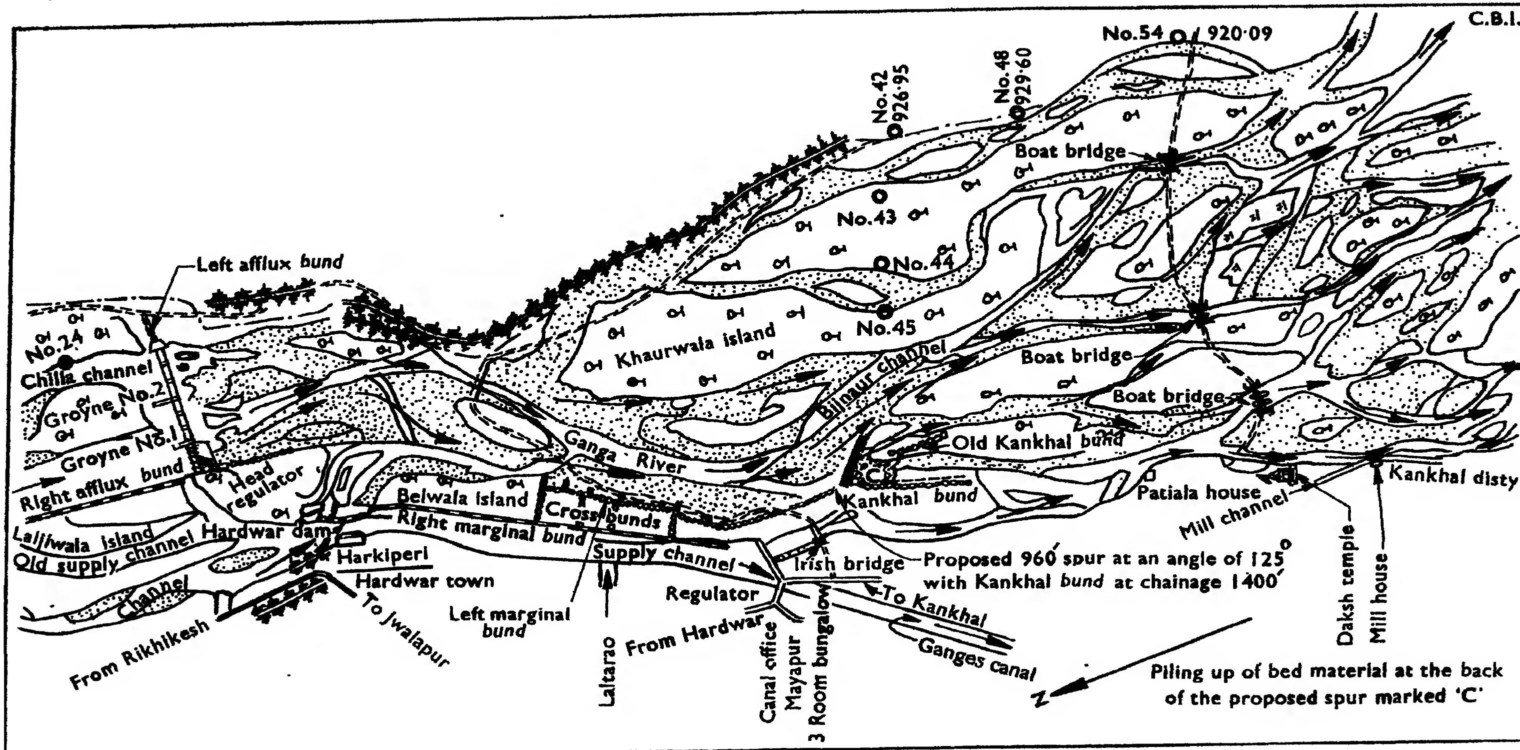
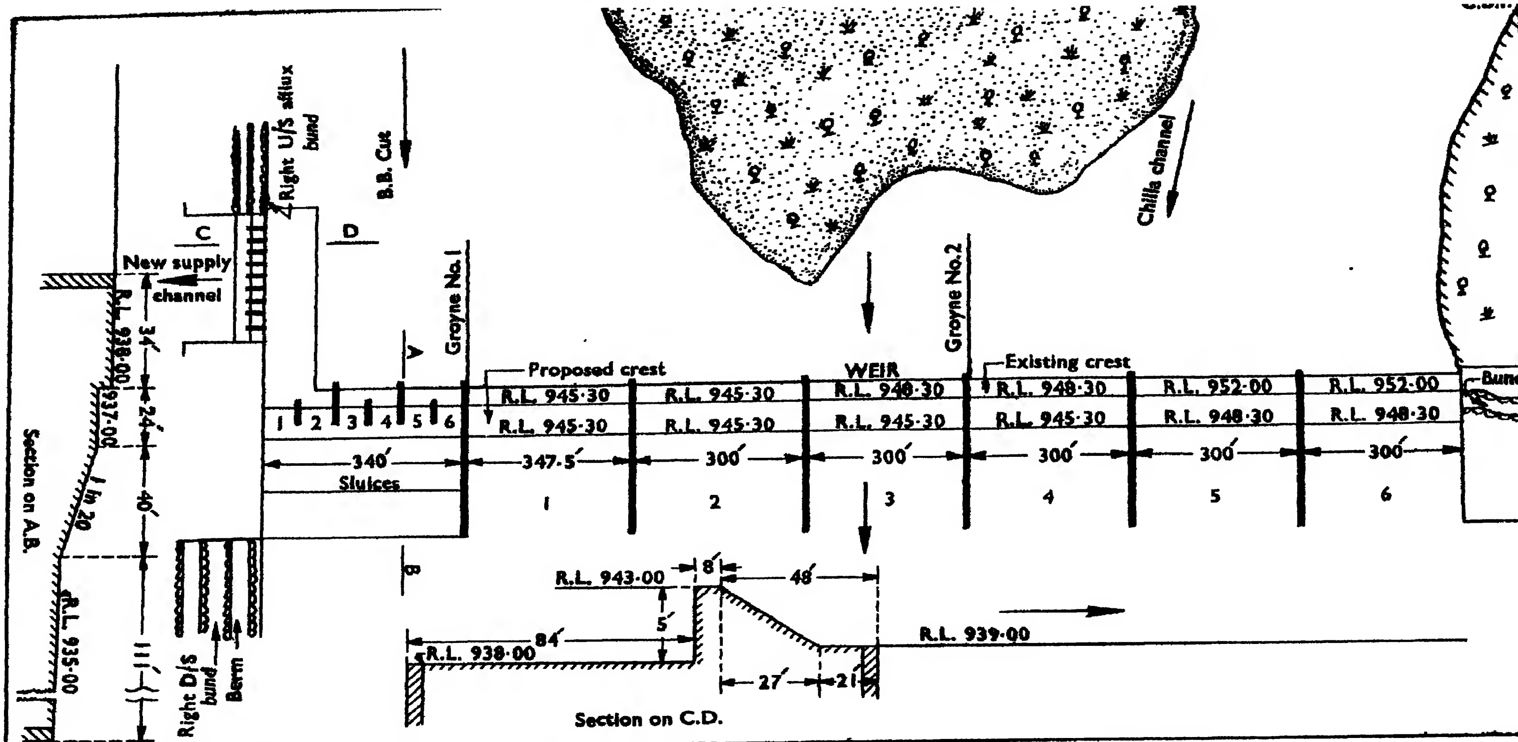


Figure 2C.2 :- *Showing the condition of flow with flood upto* 3·5 lakhs *with proposed 960′ spur at an angle of 125° with Kankhal* bund *at chainage 1,400′*

Scale 0·5 0 Mile

Reg. No 2737 XDD (C)'50 - 1,210 (P.Z.O.).



Reg. No. 2737 XDD (C)'50–1,210 (P.Z.O.).

Figure 2C.3 :- *Plan of Bhimgoda weir showing proposed modifications of crests for equitable flow*

Scale (Plan) 300 200 100 0 300 600 900 Feet

Scale (Section) 10 5 0 10 Vertical; 50 25 0 50 Feet Horizontal

(*iii*) An attempt was now made to develop Bijnor channel directly with the help of a spur by diverting the water of Kankhal channel at the junction. There was still some swing of the current towards Kankhal resulting in severe scour at the nose of the proposed spur. A parallel flow of some intensity was also obtained along the abandoned old Kankhal *bund*. However, the curvature was reversed sufficiently to throw the bed load towards Kankhal channel and choke it to a great extent. But, this position of the spur looked undesirable in view of the extreme tightening of the river section which caused both severe action and afflux.

(*iv*) The above experiment suggested a more suitable place for this spur shown in Figure 2C. 2 taken from chainage 1,400 of the *bund* and kept almost in the same direction as in (*iii*) above. The length was also reduced from 1,200 feet to 960 feet and the head of the spur reached the point of contra-flexure of the curve resulting in bringing the Bijnor channel at the outer curve. On running the model with this spur, the bed load was visibly piled up against the old Kankhal *bund* and also entirely filled up the gap between the spur and the old *bund*. Some spill continued which helped to raise the water level behind the spur and thus in spite of increasing floods, the difference between water levels upstream and downstream of the spur did not change materially. The difference in water levels was only about 10 feet upto floods of three *lakhs* and at 670,000 this increased to 16·5 feet.

When testing this device for 670,000 the flood level got abreast of the existing top of the *bund* (R.L. 949·0) at chainage 1,400 at about 5 *lakhs* and thereafter it started overflowing the *bund* below chainage 1,000, the water level upstream of the spur being R.L. 945·5 at seven *lakhs*. The top of the existing bund upto chainage 1,400 and that of the proposed spur may, therefore, be kept at R.L. 948, while the nose of the spur can be kept at R.L. 944 as there was considerable slope in the water surface along the spur.

An additional advantage, besides a thorough choking up of the gap between the spur and the old *bund*, was that it seemed utterly unnecessary to repair this old *bund*. Only at floods higher than 3½ *lakhs* the downstream nose of the *bund* came slightly into action and the *bund* started overtopping. But the bed material still continued piling up along the old *bund* thus making it difficult to breach in spite of overtopping. The nose downstream of this old *bund* is already partly armoured with concrete blocks and it would be advisable to repair the noses and fill up small gaps in the *bund* to avoid spills at medium high floods.

As a result of running seven *lakhs* of flood it was seen that the main stream got straightened both upstream and downstream of the proposed spur. The Bijnor Channel continued to develop up to 3½ *lakhs* but later on a new central

channel was scooped out. However, on subsequently running the model with lower floods, the tendency of recovering the old curvature was apparent.

RECOMMENDATIONS

(1) As a result of model investigations, it is suggested that Bijnor channel must be developed at all costs to prevent the Kankhal town from floods. A 960 feet long spur as per Figure 2C.2 is recommended to divert the flow of Kankhal channel into Bijnor channel from which the materials for this spur should be quarried. Severe action should be anticipated at the nose of this spur and the scour can be as much as 30 feet. The spur should be designed for a head of 20 feet although the maximum difference on the upstream and downstream of the spur obtained in the model was 16·5 only.

(2) With this device, the volume of water going towards Kankhal will be comparatively very little, say 5 to 10 per cent. As such it is anticipated that there shall be no action either at Patiala House or at Daksh Temple. No protective works are, therefore, recommended for these sites for the present, if this spur is built.

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(2) TO TRAIN RIVER GANGA UPSTREAM OF BHIMGODA WEIR FOR EQUALISING THE FLOW ON WEIR CREST AND PREVENTING RETROGRESSION DOWNSTREAM (7)

ABSTRACT

After the construction of present head work in 1920 it was considered desirable to bring the main stream upstream of the wave to the right to avoid parallel flow during floods as the sluices and the new supply channel were situated on the right. Gives detail of model experiments carried out to train the river for this purpose.

The weir consists of six sluices each 50 feet long and six bays of open crest each 300 feet long, on the right flank, *vide* Figure 2C.3. The levels of the record high flood of 670,000 obtained in 1924 were R.L. 962·25 upstream and R.L. 987·57 downstream at the sluices. At the left flank it was R.L. 964·5 upstream of the weir. At this time R.L. of sluices was 936 and weir crest 945·3 all along except that bay No. 6 has been raised to R.L. 948·3 in 1922.

(7) United Provinces Irrigation Research Station, Report on Research Progress during 1947, pages 54—57.

After the floods of 1924, a cut in front of the sluices known as B.B. cut was made along the right afflux *bund* in 1926 to attract the deep channel towards the sluices. Simultaneously the bays No. 5 and 6 were raised to R.L. 952 and Nos. 3 and 4 by three feet *i.e.* to R.L. 948·3 against the sluice R.L. 938. This ultimately forced the B.B. cut to develop into a straight deep channel in 1927 and the old main stream opposite bays No. 5 and 6 known as Chilla channel got silted up gradually. This worked very well for a few years but soon the entire stream got diverted to the sluices and the discharge across the weir dwindled very considerably. Evidently the raising of bays No. 3, 4, 5, and 6 has been overdone. Another thing for increasing the retrogression was the raising of the upstream floor at the sluices to R.L. 939 in the year 1944 which created increased drop at the sluices. The concentration of river at the sluices has started creating serious difficulties during the past few years.

MODEL INVESTIGATIONS

On the model Figure 2C.1, with levels of the crest as existing at present, floods of different intensities up to six *lakhs* were run and it was found that the Chilla channel would not develop in any case. The channel bifurcating from the B.B. cut towards bays No. 5 and 6 on the left indicated some shoaling tendency *vide* Figure 2C.1.

On inspecting Chilla channel it was found that it was full of fine silt and sand thus indicating that it was taking only surface waters from the outer curve at its head. Accordingly it was feared that bays No. 5 and 6 in front of Chilla channel must not be lowered too much. It was thus decided to lower the crests of bays No. 5 and 6 by 1·5 feet only in the first instance. Several cycles of floods were run and the Chilla channel showed no signs of development. Only slightly better flow started in the channel taking off from the B.B. cut on the left.

Crest of bay No. 3 was next lowered to R.L. 945·3 (R.L. of bays No. 1 and 2) and Nos. 4, 5 and 6 kept at R.L. 948·8. This passed better supplies in bay No. 3 only from the B.B. cut. The channel approaching Nos. 5 and 6 did not show much improvement. The afflux between left and right flanks upstream which was previously about 3 feet at 1·5 *lakhs* was now only 1·25 feet. This indicated an increase in the discharge per foot from 24 cusecs to 44 cusecs in bays No. 5 and 6. The discharge in bay No. 4 was, however, considerably reduced compared to what it was when bays No. 5 and 6 were at R.L. 952 being now only 44 cusecs against 60 cusecs per foot previously. This also resulted in shoaling upstream of the bay No. 4. Thus taking the average of all the three bays (4, 5 and 6) the average discharge improved only from 36 cusecs per foot run to 44 cusecs per foot run at floods of 1·53 *lakhs, vide* Table 2C.3.

Superintending Engineer, I, on inspecting the model, suggested shifting of groyne between bays No. 3 and 4 to the pier between bays No. 4 and 5, as that might help cutting the island above in low supplies. On trial this was

TABLE 2C.3

Statement showing distribution of discharge over weir studied for a discharge of 153,000 for different adjustment of crest levels

	Original conditions				After proposed modifications as per Experiment No. 1 below					After proposed modifications as per Experiment No. 2 below					After proposed modifications as per Experiment No. 3 below				
Bays No. and Sluices	R. Ls. of crests	Discharge per foot in cusecs	Per cent discharge passing without excluder	Afflux on left flank.	R. Ls. of crests	Discharge per running foot	Per cent discharge passing with excluder	Afflux on left flank	Remarks	R. Ls. of crests	Discharge per running foot	Per cent discharge passing with excluder	Afflux on left flank	Remarks	R. Ls. of crests	Discharge per running foot.	Per cent discharge passing with exceuldr	Afflux on left flank	Remraks.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Bay																			
1	945·3	67	53	3·0	945·3	66	..	1·25	U.S. Groyne in front of pier No. 3 as exting	945·3	66	60	1·25	Groyne in front of pier No. 4. It masked Bay No. 4	945·3	65	67	1·0	Groyne in front of pier No. 3 as existing. There was dead region around the groyne.
2	945·3	67			945·3	66				945·3	66				945·3	65			
3	948·3	33			945·3	66	64			945·3	66				945·3	65			
4	948·3	60			948·3	44				948·3	24				945·3	81			
5	952·0	24			948·3	44				948·3	44				948·3	44			
6	952·0	24	47		948·3	44				948·3	44	40			948·3	44			
Sluices 1 to 6	939·0	240			939·0	185	36			939·0	205				939·0	151	33		

found to screen bay No. 4 completely, reducing its discharge to 24 cusecs per foot run against the original of 60 cusecs. This bay was, as usual, getting its supplies from the left channel only and thus resulted in reducing the average discharge from bays No. 4, 5 and 6 to 36 cusecs as originally. The present position of the groyne was thus left untouched and it was considered necessary to lower bay No. 4 to R.L. 945·3 to create better draw from the eastern channel. The position improved considerably and the island upstream showed signs of being cut away gradually due to increased velocities on either flank. The average discharge per foot from bays No. 4, 5 and 6 now became 56 cusecs. There was still water at the groyne between bays No. 3 and 4.

The distribution of flow across the weir was now much better, as apparent from the comparative statement given *vide* Table 2 C.3 for a discharge of 153,000 cusecs.

RECOMMENDATIONS

(1) From the above, it is clear that the relief that would accrue at the sluices with the alterations suggested would mean reducing the flood percentage from 47% to 33%. It is considered that this would by no means prove any drastic change in the regime of the river upstream and may be adopted in the first instance. Any modifications to this can be studied after experiencing the floods for a year or two.

(2) The crests have been so lowered that the existing gates of bays No. 3 and 4 can be utilized in bays No. 5 and 6 and a fresh set of gates may be got prepared for bays No. 3 and 4.

(3) A length of 32 feet crest blocked in bay No. 1 may also be knocked off which is wholly responsible for a dead region just below the fish ladder and will further help in equitable distribution of discharges across the weir, unless required for the protection of the crane house.

(3) MODEL INVESTIGATION FOR THE PROPOSED MOR BARRAGE (8)

ABSTRACT

In the Mor Project, a barrage was proposed to be constructed on the Mor River at a site called Tilpara Ghat near Suri in the district of Birbhum in Bengal. The design of the structure and its tentative position in the river were suggested by the Superintending Engineer, Special Duty, and the problem was referred to the Institute for investigation in the models for decision on the following points:-

(1) The exact selection of the site, position and orientation of the barrage.

(2) Training Works—Design of guide banks, bank and bed protection, design of divide walls.

(3) Methods of regulation of the undersluice and barrage gates.

(4) Silt excluding devices, if necessary.

(5) Testing the proposed sections for uplift pressure and surface flow.

(6) Determination of the necessary downstream protection like friction blocks on the downstream floor.

(8) River Research Institute, West Bengal, Annual Report, 1947, pages, 62-67.

It was decided that the items (1) to (4) above were to be tried in a river model, and a part river model on a bigger scale for item No. (4) above, if found necessary. Item Nos. (5) and (6) were to be investigated in sectional models in a flume. Gives results of experiments.

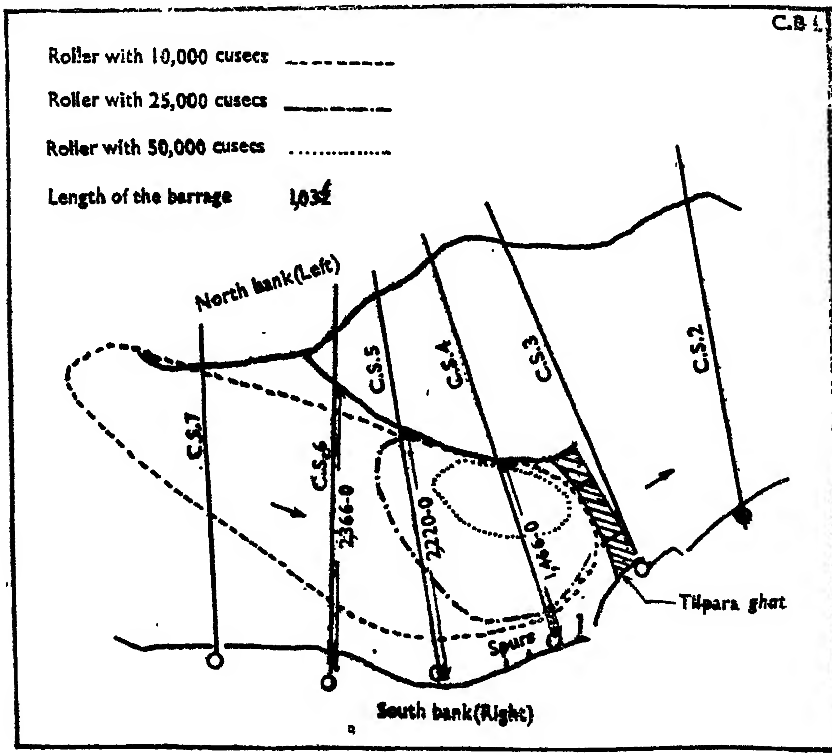

Figure 2C. 4: *Showing the Survey. Plan of More River.*

MODEL AND EXPERIMENTS

This model was constructed and worked to investigate the points (1) to (4) mentioned above. Work done on this model during 1946 was published in the Annual Report of the Institute for 1946 (Publication No. 2). As mentioned in the last paragraph on page 32 of this publication, the authorities in-charge of the construction of the barrage found on further investigation that it would be possible to construct the barrage at cross section No. 3 Figure 2 C.4, without removing any of the memorials. As the barrage at cross section No. 3 itself was found to behave better than at 120 feet dowstream of cross-section No. 3 (mentioned in 1946 Annual Report) it was necessary to try different orientations of the barrage at cross-section No. 3 and find out the best position. The river model was first built on horizontal scale $\frac{1}{400}$ and vertical scale $\frac{1}{80}$. These scales were chosen because they were found to work well in the case of the river model of the Damodar, a flashy river (shallow and wide) like the More But

it was found that bed movement near the barrage in this model would not start with discharge below 100,000 cusecs. The canals, however, would be closed in the prototype when river discharge would go up above 100,000 cusecs. The problem of silt entry into the canals could not therefore be studied in this model. In January 1947, when the first part of the experiment was completed (published in the annual report of 1946), it was decided to change the vertical scale of this model to $\frac{1}{30}$ so that bed movement might be initiated at lower discharges. In fact, when the model was run after changing the vertical scale to $\frac{1}{30}$ bed movement near the barrage was observed even at the discharge equivalent to 25,000 cusecs in the prototype, which is the average high discharge of this river. The canals would be kept open in the prototype at this stage and it was therefore thought possible to study the silt exclusion problem in this model. The change of the vertical scale from $\frac{1}{60}$ to $\frac{1}{30}$ revealed an important feature of the limitations of model experiments. With the barrage in position No. 4 (Ref. page 29 of 1946, Annual Report—barrage at cross-section No. 3 with right head regulator flush with the right bank and centre line of the barrage making an angle of 6° 30′ with cross-section No. 3 towards the upstream) a surface roller was observed above the barrage when discharge was equivalent to 10,000 cusecs in the river. No such roller was observed at this river discharge when the vertical scale was $\frac{1}{60}$. This roller is produced by the transfer of momentum from the advancing column of water on the right bank to the stagnant mass of water on the left. With increase of scale the magnitude of the momentum transferred increases and goes above the minimum value necessary to put the stagnant mass of water on the left in motion. This explains why the roller was not observed when the vertical scale was $\frac{1}{60}$, but came in view when the vertical scale was inceased to $\frac{1}{30}$. It follows that with increase of scale this minimum value is attained with lower discharges, so that the roller is expected in the prototype at a much lower discharge than at the stage observed in the model.

The design of the barrage was slightly modified at this stage by the Designs Office by reducing the barrage waterway by 60 feet and raising the whole structure by two feet. The barrage was constructed according to this altered design and put in the model. The total length of the barrage between the inside of two head regulators was 1,032 feet, the crest level of the undersluice section was at R.L. 192·0, the crest level of the weir section was at R.L. 195·0 and the sill level of the head regulators was at R.L. 199·0. The pond level to be maintained above the barrage was as before at R.L. 207·0. The following three orientations of the barrage line at cross-section No. 3 were tried in the model.

(1) Barrage line making an angle to 15° with cross-section No. 3 towards the upstream with the right head regulator *flush with* the right bank at cross-section No. 3.

(2) Barrage line making an angle of 6° 30′ with cross-section No. 3 towards the upstream with the right head regulator *flush with* the right bank at cross-section No. 3.

(3) Barrage line placed along cross section No. 3 with the right head regulator *flush with* the right bank at cross-section No. 3.

The conditions of flow for case (2) above was found to be the best. To avoid the upstream nose of right head regulator project inside the river bed the inside face of this head regulator was constructed to make an angle of 100° with the centre line of the barrage. This worked well.

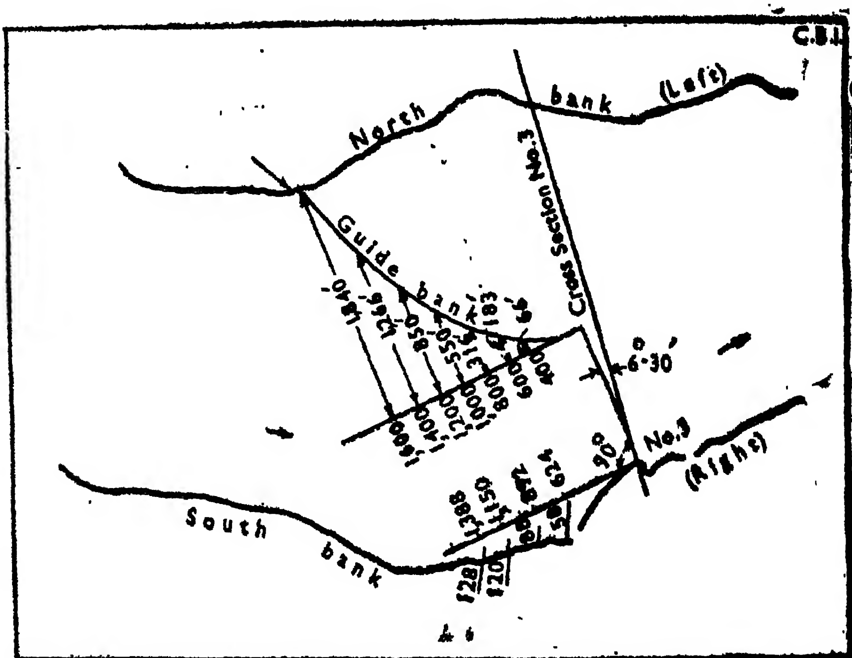

Figure 2 C.5: *Survey Plan of River Mor showing the conditions of flow with Barrage line making different angles with cross-section No. 3.*

The best shape of the left guide bank was found after some adjustment. The length and position of spurs on the right bank above the barrage, found previously (described in 1946 Annual Report) was found to work satisfactorily. The whole arrangment is shown in Figure 2 C.5. This was found to be the best arrangement with the barrage at cross-section No. 3.

The undesirable feature of this site was that for the average discharge of the river, the whole flow was concentrated within the right 400 feet of the barrage, and the whole volume of water within 4,000 feet above the barrage except this narrow strip along the right bank was more or less a stagnant pool. The following devices were tried in the model to distribute the flow over the barrage more uniformly and initiate forward movement over the whole width of the pond :—

(1) Pitched islands above the barrage.

(2) A cut from cross-section No. 7 to the left undersluice of the barrage.

(3) A spur at the right bank at cross-section No. 6 in combination with an initial cut from the right bank to the left undersluice as in (2) above.

None of these devices produced the desired effect. Pitched islands did not come into operation except when the discharge in the river was high. This happened very rarely not even for a few days in every year. The cut was filled up with sand as soon as the discharge in the river increased.

It was also found in the model that the silt entering the left bank canal was nearly ten times more than that entering the right bank canal. It was noticed that silt would deposit in the left pocket in the front of left canal head regulator to be picked up from there into the canal.

The different positions of the barrage tried so far were confined within the reach between cross-section No. 2 and cross-section No. 3 as this reach was considered to be suitable from the construction point of veiw. The right bank below cross-section No. 2 was rather low. But the best position within this reach as found above still had the following undesirable features :—

(*a*) A slow moving roller was seen to persist just above the barrage at low discharges of the river. This is likely to lead to the formation of an island just above the barrage leaving a waterway of only about 400 feet width along the right bank. This construction will be dangerous when a flood above 50,000 cusecs comes over the barrage.

(*b*) A flow parallel to the barrage from the right pocket to the left also persists and this may lead to excessive silt entry into the left bank canal and heavy scour at the nose of the divide walls.

The operation of the model in this position was shown to the Chief Engineer on September 25, 1947. He desired that the matter be referred to the Consulting Engineer to the Government of India, Central Waterways, Irrigation and Navigation Commission who inspected the model experiments on October 7, 1947 and October 23, 1947 and decided in favour of a site 2,750 feet below cross-section No. 3, in spite of some constructional disadvantages there. The following two positions of the barrage at this lower site near cross-section No. 2A were tried in the model :—

(1) The right head regulator pushed 200 feet into the river bed and the barrage line making an angle of 17° 45′ with cross-section No. 2A towards the upstream.

(2) The right head regulator kept flush with the right bank and the barrage line made normal to the right bank line joining the bank points at cross-section No. 3 and the barrage site. The right bank from the right head regulator to cross section No. 3 was made straight by removing the bank at one place and filling at another place. The

total earth work was less than 500,000 cubic feet. The right guide bank was to follow this bank up to the cross-section No. 3.

The following levels have been fixed by the Design Officer for placing the barrage in this position :—

(*a*) Crest level of the undersluice section=R.L. 190·5

(*b*) Crest level of the weir section=R.L. 193·5

(*c*) Sill level of the canal head regulators=R.L. 198·75

(*d*) Pond level to be maintained=R.L. 206·75

In position (1) above the flow over the barrage at high discharge was not uniform. The main flow during high discharges was found to pass over the right two-thirds of the barrage and the intensity of flow over the left third was comparatively much less. To remove this defect the right bank canal regulator was made flush with the bank and the barrage line tilted to be at right angles to the line joining the bank points at cross-section 3 and the upstream of the right regulator. The shape of the left guide bank was adjusted so as to lead the current smoothly along it at high stages without any attack at any point on it. The barrage in position (2) above with the guide banks is shown in Figure 2C.6. With this position of the barrage there was wider passage of straight flow along the right bank expanding from 500 feet at cross-section No. 3 to 800 feet at 400-feet upstream of the barrage line whose total width was 1,032 feet. The left undersluice pocket only was within the stagnant water

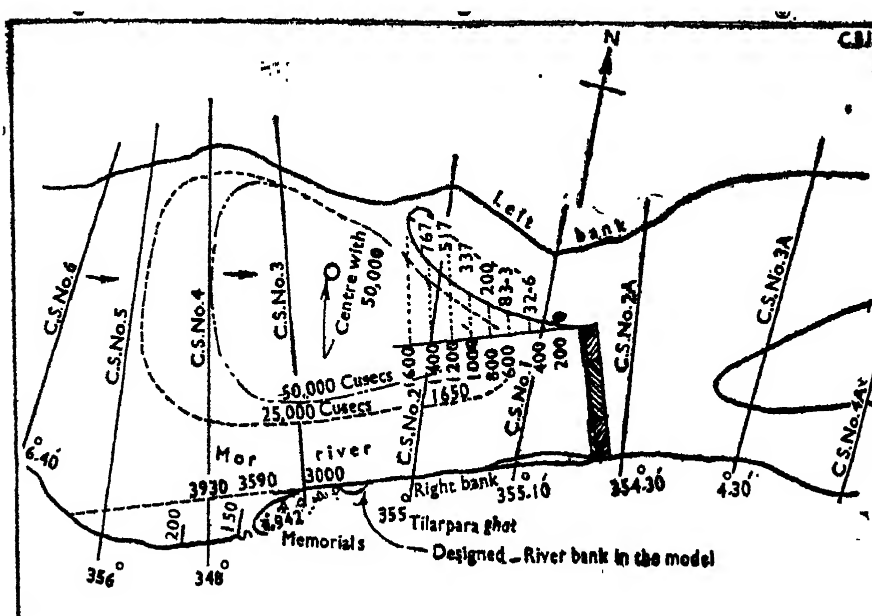

Figure 2C.6: *Plan of the River Mor near Tilpara Ghat showing the position of guide banks.*

zone, but even there the bottom flow was observed to be towards the pocket. The zone of forward flow was marked nicely by mustard seeds depositing on the bed on the left edge of the channel. (These mustard seeds were thrown from cross-section No. 9, about 1½ mile above barrage, while a steady discharge of 25,000 cusecs was passing over the model for 8-hours), Figure 2C.7. The zone of surface roller and a stagnant water, Figure 2C.6 was fairly above the barrage and any high island forming within this zone would not be as dangerous to the barrage as when it was at cross-section No. 3. This site, Figure 2C.6 was, therefore, considered to be better than the site at cross-section No. 3 shown in Figure 2C.5.

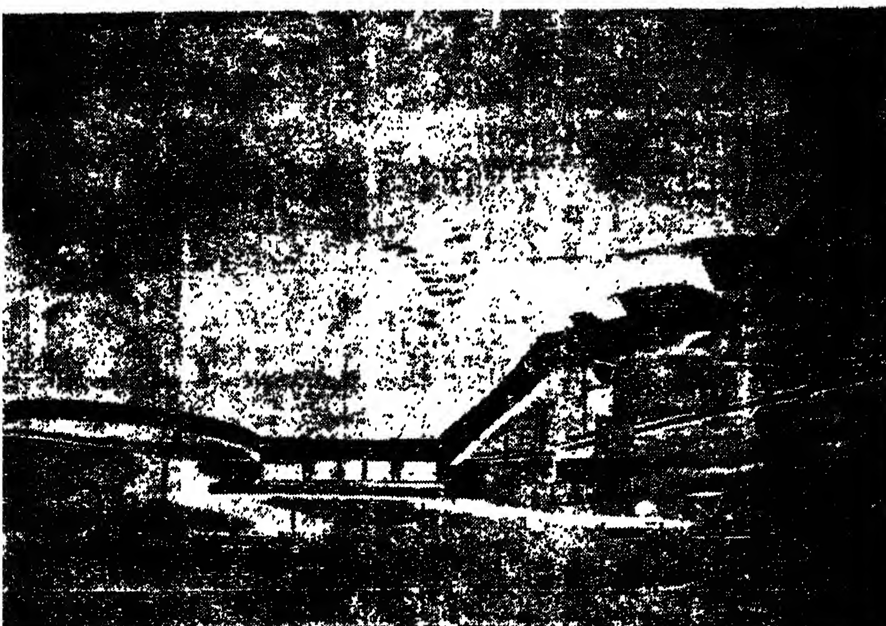

Figure 2C.7 : *Showing the lines of seed deposit on the bed after eight hours run with 25,000 cusecs (Mor River Model.)*

It was found that the right bank at cross-section No. 6 and cross-section No. 3 was attacked by the river at all stages. To protect the bank at cross-section No. 6 the bank was pitched with stone chips from 200 feet above to 200 feet below the cross-section point. It was found that unless the right bank at cross-section No. 6 is held, attack at cross-section No. 3 will increase. To divert the attack on the right bank at cross-section No. 3, the following systems of spurs were tried :—

(1) One downstream spur of 100, 150 and 200 feet respectively at cross-section No. 6.

(2) One T-head spur at cross-section No. 6, projecting 150 feet into the river bed.

(3) One downstream spur 150 feet long put up at a point 300 feet below cross-section No. 4.

(4) One downstream spur 150 feet long as in (3) above and one downstream spur 150 feet long placed at a point 230 feet above cross-section No. 4.

(5) The two spurs of (4) above with another downstream spur 80 feet long placed at a point 350 feet above cross-section No. 3.

(6) The two spurs of (4) above with a T-head spur in place of the 3rd spur of (5) above.

The results of experiments showing comparative silt entry into the canals are given in Table 2C.4.

TABLE 2C. 4

Date of experiment 1947	Discharge 25,000 cusecs run over the barrage in the final position 2,750 feet below cross-section No. 3	Quantity of mustard seeds in grain		
		Dropped at C. S. No. 9	Entering the Right Canal	Entering the Left Canal
November 3	Without any spur	3,600	35	410
November 18	With a spur 150 feet long at C. S. No. 6	3,600	22	469
November 21	With a spur 200 feet long at C. S. No. 6	3,600	44	119
December 1	With a T-head spur projecting 150 feet from the right bank at C. S. No. 6 ..	3,600	20	210
December 5	With a spur 150 feet long at a point 300 feet below C. S. No. 4	3,600	5	90
December 13	With a spur 150 feet long at a point 300 feet below C. S. No. 4 and a spur 150 feet long at a point 230 feet above C. S. No. 4	3,600	16	178
December 31	With two spurs as in above and another T-head spur at a point 300 feet below C. S. No. 4	3,600	30	100

(4) PROTECTING THE RIGHT BANK OF THE KUAKHAI RIVER ABOVE THE BENGAL NAGPUR RAILWAY BRIDGE NEAR CUTTACK (ORISSA) [9]

ABSTRACT

The Kuakhai takes off from the Katjuri River on the southern side of Cuttack. The Bengal Nagpur Railway crosses the Mahanadi, the Katjuri below Kuakhai head, and the Kuakhai. These railway bridges are not provided with guide banks. Upstream of the Kuakhai head, the Katjuri spills over a vast stretch of low area known as the Banki Road spill. This flood spill begins to flow when the total Mahanadi flood above Naraj is about 10 *lakhs* causecs and finds its way mainly into the Kuakhai above the railway bridge and to a very small extent flows downstream of the railway embankment through culverts. Describes experiments carried out to protect the right bank of the Kuakhai.

The Orissa Public Works Department had observed the discharges of the Kuakhai at the railway bridge in 1939 and 1940, when the floods of the parent Mahanadi had risen to 13·61 *lakhs* cusecs and 14·545 *lakhs* cusecs respectively. Based on these, the discharge of the Kaukhai, with a 15·5 *lakhs* flood in Mahanadi works out to 2·66 *lakhs* cusecs, which is the same as indicated by the Mahanadi model. Rihind had computed discharge figures of the various branches of the rivers for the 1872 flood conditions, but these figures may not now be applicable ; (*a*) because of the various 1872 breaches in the embankments in the Naraj gorge, and also at the head of the Katjuri ; (*b*) because the Barang *Nala* taking off from the right bank below Naraj no longer exists ; and (*c*) because of the aggradation of bed levels and changes in channel sections that have occurred since.

Though the history of the scour along the right bank of Kuakhai, upstream of the railway bridge, is not available, yet it appears that the trouble is a chronic one and was met in the past by providing short term protection in the shape of stone pitching as and when required. There now exist two discontinuous pitched banks one being at right angles to the railway bridge and the second curved bank projecting beyond the normal alignment of a guide bank with a gap in between the two, probably to admit water from the small *nala*. This *nala*, as seen from the topo sheet, has no regular catchment and appears to have formed by the scour caused by the Banki spill waters. In recent years there has been serious erosion at the upstream end of the curved, detached bank.

THE MODEL

After extending the Katjuri on its right bank to the railway embankment the existing, 1/400:1/66 vertically exaggerated model of the Mahanadi and its branches was used for these experiments. This model was a ready asset in the speedy solution of the problem, because the model had correct entry and exit conditions. At the beginning of each experiment, the model was relaid to 1943 conditions and flood cycles rising from 6 *lakhs* to 15 *lakhs* and falling back to

(9) Central Waterways, Irrigation and Navigation Research Station Poona, Annual Report Technical, 1947, pages 39-42.

6 *lakhs* above Naraj were repeated till stable conditions were obtained.

EXPERIMENTS AND DISCUSSION OF RESULTS

Experiment No. 1 with existing conditions.—In this experiment, the discontinuous stone pitchings on the right bank at the bridge were reproduced. The curved projecting portion of the pitching acted like an attracting spur as expected. It was orginally considered that the Banki spill, flowing into the Kuakhai, might be the cause of eddies and turbulence and thus be responsible for the scour. Visual observations, however, indicated that the attack at the nose was severe when the Banki spill was not flowing.

Having reproduced the prototype conditions in a preliminary run, 'repelling spurs' 1,200 feet long (where river width about 3,000 feet) with their shanks raised above high flood level and joined to the railway embankment were tested, 2,500 feet to 4,000 feet upstream of the bridge, but were not seen to be satisfactorily effective in diverting the flow away from the right bank at the bridge.

A set of two spurs, *viz.*, one 'attracting spur' on the right bank and the other 'repelling spur' on the left bank at the offtake of Kuakhai, were also tested to centralize the flow through the bridge. The lengths of the spurs were varied from 400 feet to 1,200 feet; but they, too, did not prove satisfactory in that the 'attracting spur' being on the convex (*i.e.*, sanding) side did not act as an attracting spur; and, also, in that the conditions at the bridge did not improve.

In Experiment No. 2, therefore, a continuous stone pitched guide bank was tested, Figure 2 C.8

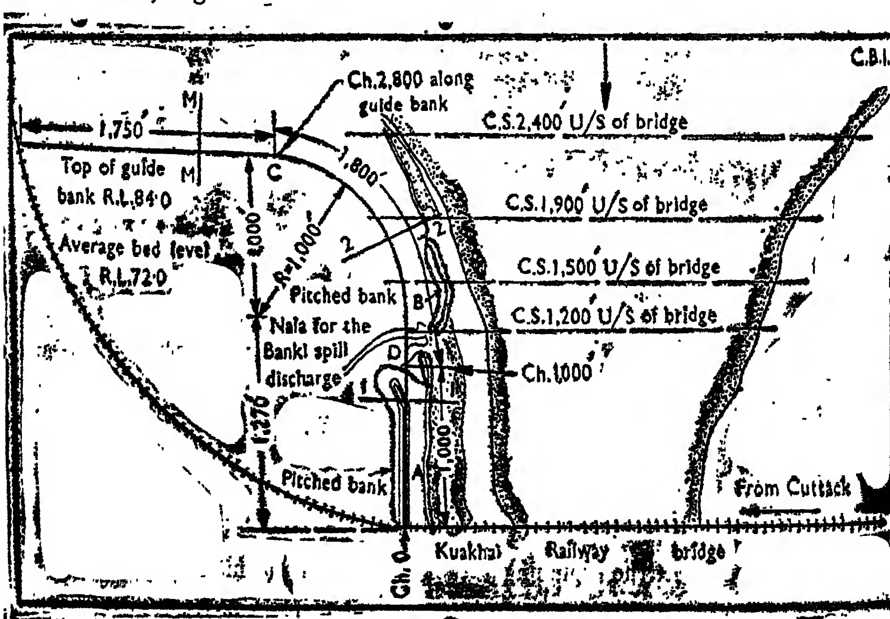

Figure 2 C.8 : *showing the alignment and length of the pitched guide bank. (Experiment No. 2)*

LENGTH OF THE GUIDE BANK

Generally, according to the widely accepted practice, the length of a guide bank is made equal to the length of the bridge. In the present case, however, the railway bridge was considered to be much too long for the following reasons :—

Table 2 C.5 shows the data of the railway bridges across the Katjuri and the Kuakhai.

TABLE 2 C.5

Serial No.	Items	Katjuri Railway Bridge	Kuakhai Railway Bridge
1	Q (discharge in cusecs)	5·616 *lakhs* cusecs (extrapolated from observed figures of 1939 and 1940)	2·66 *lakhs* cusecs (extrapolated from observed figures of 1939 and 1940)
2	W =(Length of the bridge)	2,700 feet	3,000 feet
3	C in $W=C\sqrt{Q}$ ..	3.60	5.82
4	Lacey W *i.e.* $W=2{\cdot}67\sqrt{Q}$	2,000 feet	1,373 feet

Comparing the 'C' values in item 3 of Table 2 C.5, it is considered that, compared to the Katjuri Railway Bridge, the Kuakhai Railway Bridge is $1{\cdot}62 \left(=\frac{5{\cdot}82}{3{\cdot}60}\right)$ times too long for the discharge it has to pass.

Even allowing for the fact that the Kuakhai lies on the convex arm of the branch and would therefore be wider by virtue of "sanding", it would appear sufficient to assume the Kuakhai bridge 20% longer relatively, *i.e.*, having a C value of 4·4 in $W=C\sqrt{Q}$ against 3·6 for the Katjuri and 5·82 for the existing, inordinately long Kuakhai bridge. The guide bank was thus kept 2,270 feet long.

The guide bank was connected to the railway embankment after rounding off its nose with 1,000 feet radius, 90° curve.

FLOW CONDITIONS

Flow conditions with this guide bank were markedly better than under existing conditions.

Though with low floods, the flow hugged the guide bank, yet the high velocity filaments were away from it ; and when the Banki Road spill began to flow with floods higher than about 10 *lakhs* (above Naraj), the spill discharge was smoothly guided along the shank of the guide bank joining the curved head to the railway embankment. With the peak flood, the high velocity filament, which was near the curved head, gradually moved away from the guide bank so that at the bridge it was nearly 400 feet away from the guide bank. It was observed that round the curved nose of the embankment, the scour was less than under existing conditions ; because the flow was smoothly guided along the 1,000 feet radius curve. Between chainages 1,000 feet to about 1,800 feet, however, though the scour depth remained the same yet a deep channel formed along the toe of the embankment. This, however, is quite natural due to the attracting effect of a guide bank ; and with a properly designed

falling apron, the condition will be well under control. It is for this reason that the reach between chainages 1,000 and 2,800 feet is proposed to be provided with a launching apron sufficient for a scour depth of $2D_{\text{Lacey}}$.

DESIGN OF THE GUIDE BANK

As plenty of sand is available at site, the guide bank may be made with a sand core—width at the top=6 feet and having 3 feet thick, one-man stone pitching on the side slopes. As a measure of safety, it is essential to lay a launching apron sufficiently thick and long to launch and line scour-holes that may develop along the guide bank.

According to the model observations, scour hole is expected to be deep in portion (1) from chainages 1,000 feet to chainages 2,800 feet of the embankment, being shallower beyond chainage 2,800. Hence, the launching apron may be provided to be adequate for a maximum depth of—

$1 \cdot 5\ D_{\text{Lacey}}$ or 35·05 feet, say 35 feet, from chainage 0 to 1,000

$2D_{\text{Lacey}}$ or 46·74 feet, say 47 feet, from chainage 1,000 to 2,800

D_{Lacey} or 23·37 feet, say 25 feet, beyond chainage 2,800

(where D Lacey$=0 \cdot 47\ (Q/f)^{\frac{1}{3}}$; $f=1 \cdot 76\sqrt{m}$ and $Q=2 \cdot 66$ *lakhs* cusecs and $f=1 \cdot 96$).

With H. F. L. 79·4, the deepest bed will be at
R. L. 44·4 between chainage 0 and 1,000
R. L. 32·4 between ,, 1,000 and 2,800
R. L. 56·0 beyond ,, 2,800

TABLE 2 C.6

Chainage from Bridge	Average bed R. L.	Length in feet along the sloping face of the scour-hole from R. L. in column 2 to bottom (with 2 : 1 slope)	Quantity of stone required *i.e.*, column 3 × thickness of launched apron (=3 feet) in cubic feet	Size of apron required *i.e.*, width × thickness allowing 10% for some stones being washed away *etc.* in square feet
1	2	3	4	5
0 to 1,000	60·0	35	105	25×4·5
1,000 to 2,800	70	84	252	35×8
Beyond chainage 2,800	72	35·7 Say 36	108	25×4·5

The aprons, as shown in Table 2 C.6 when laid to the average bed level, the toe of the pitched embankment, should provide adequate protection to the guide bank.

The top of the apron on the high ground will be at a level higher than R. L. 72·0, whereas around the curved head, it will be two feet lower and that up to chainage 1,000 feet will be still lower. It should, however, be noted that the apron top should have a gradual slope along its length so as not to present sudden drops at the points of change-over of levels.

DOUBLE ACTING SLUICE

Unless the guide bank is given a designed section to withstand about 12 feet head on one side only, it will be necessary to provide for an adequate opening to fill in the compartment formed between the railway embankment and the guide bank.

The data for the design of such a two-way sluice are as follows :—

(1) Area of compartment $=496\times10^4$ sq. ft.
(2) Average bed R. L. $=72\cdot0$
(3) H. F. L. to which it should be filled up $=79\cdot4$
(4) Volume of water to be impounded in the compartment $=496\times10^4\times(79\cdot4-72\cdot0)$
$=3\cdot66\times10^7$ cft.
(5) Average rise of water upstream of the guide bank $=2\cdot81$ ft. per day
(6) Average time in which the compartment should be filled $=2\cdot78$ days.

Working on this data, the average discharge the sluice has to pass so that the water level in the compartment will be the same as that on the upstream side of the guide bank, will be

$$=\frac{3\cdot66\times10^7}{2\cdot78\times24\times3{,}600}$$

$=152$ cusecs

The head available to pass this discharge varies from zero to $7\cdot4$ feet (R. L. $729\cdot4$—720). Hence working on an average head of $\frac{7\cdot4}{2}$ *i.e.*, $3\cdot7$ feet and assuming that under drowned conditions and a flat sill, the coefficient of discharge will be of the order of $2\cdot0$, the length of the opening required will be

$$=\frac{Q}{c\times D^{1.5}}$$

$$=\frac{152}{2\times3\cdot7^{1.5}}$$

$=10\cdot67$ *i.e.*, say 11 feet

As the discharge to be passed through the sluice is small, a simple masonry wall 5 feet to 6 feet deep with its top at the average bed R. L. and with its upstream and downstream sides pitched with 3 feet thick stone pitching is expected to serve the conditions.

CONCLUSION

Under existing conditions, the erosion is due to the curved flow and is accentuated by the stone pitching which probably was not designed as a permanent measure. A guide bank, as suggested in the Note, appears to be a promising measure.

(5) PROTECTION OF THE LEFT BANK OF KATJURI RIVER AT KHANNAGAR UPSTREAM OF THE KATJURI RAILWAY BRIDGE—ORISSA(10)

ABSTRACT

During the 1946 floods, chiefly owing to the sustained flood of about 12 *lakhs* cusecs above Naraj flowing for five days at or about a stage two feet below the maximum, the attack at the right bank of the Katjuri upstream of the Kuakhai head moved downstream causing concentration of flow which scoured the right bank at Kazipatana, just downstream of the Kuakhai off-take ; the flow then was sharply deflected from the right bank at Kazipatana to the left bank near the Burning Ghat downstream of Cuttack. Further downstream, the flow hugged the left bank, causing severe damage between the Burning Ghat and the Khannagar revetment and also along the revetment for a length of 2,500 feet. Extensive emergent protection work with sand bags, brush wood and temporary piles was done to prevent the embankment breaching. Permanent protective works appeared necessary before the 1947 floods for which model tests were carried out. Describes results of experiments.

THE MODEL

These tests were carried out in the $\frac{1}{400} : \frac{1}{66}$ vertically—exaggerated model of the Mahanadi and its branches which was being successfully operated to study the redistribution of sand and discharge between the Mahanadi and Katjuri at Cuttack. The entry and exit conditions having been already proved, this model was a ready asset in the quick solution of the new problem.

Experiments : The following qualitative tests were done to arrive at the best results :—

Experiment No. 1 : with existing conditions.

Experiment No. 2 : with 1,200 feet long spur near the Burning Ghat, pointing 45° upstream.

Experiment No. 3 : with 1,200 feet long spur near the Burning Ghat, pointing 60° upstream.

Experiment No. 4 : with 1,400 feet long spur near the Burning Ghat, pointing 90° upstream *i.e.*, at right angles to the bank.

Experiment No. 5 : with 1,400 feet long spur near the Burning Ghat, pointing 60° upstream.

In each experiment a flood discharge of six *lakhs* cusecs above Naraj—which evidently gave bad conditions at Khannagar, specially with a spur, because the throw-off due to a spur increased with increased discharges, was run for three hours during which time observations of velocities, lines of flow and regions of scour and shoaling *etc.* were made.

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(10) Central Waterways Irrigation, and Navigation Research Station, Poona, Annual Report, Technical, 1947, pages 43-45.

Finally, the two most promising alternatives were tested under flood cycles rising from six *lakhs* cusecs to 15 *lakhs* and falling back to six *lakhs* cusecs (above Naraj).

DISCUSSION OF RESULTS

In preliminary tests, different lengths of spurs were tested at different sites. Spurs shorter than 1,200 feet at and upstream of the Burning Ghat did not appreciably shift the high velocity flow from the Khannagar bank. Spurs sited further upstream than shown in Figure 2 C.9 were also inefficient because, after local deflection, the flow again swung back to the left bank at Khannagar. Shorter spurs might probably have worked fairly satisfactorily, if located further downstream, but this was not practicable due to great depth of water. The best position of a spur was found to be near the temple between cross-section No. 7 and cross-section No. 8.

VELOCITIES

In Experiment No. 1 (existing conditions), the highest velocity was near the left bank, which gradually fell off to a minimum near the right bank. The effect of putting in a spur was to reduce the velocities at the left bank and increase those at the centre and a little near the right bank. Optimum normal distribution,—*i.e.*, high in the mid channel and low at the sides—was obtained with a 1,400 feet long spur pointing 60° upstream. The next best distribution was obtained with a 1,200 feet long spur pointing 60° upstream.

LINES OF FLOW

The bed and surface lines of flow in Experiments No. 1 to 5 observed with a flood discharge of six *lakhs* cusecs above Naraj showed that, under existing conditions the surface flow concentrated and dived near the Khannagar bank, the bed water being deflected away from the bank, indicating a likely tendency to undermine the bank. With a 45° spur, the throw-off of the high velocity flow was not sufficient. The 60° spur gave a better throw-off of surface flow and the return flow downstream of the spur was also less severe. A 90° spur deflected the bed flow below it away from the left bank—which was undesirable for inducing silting and the surface flow throw off was also less than with the 60° spur. The optimum results were, thus, obtained with the 60° spur.

Having observed the lines of flow and velocities, Experiments No. 3 and 5 were repeated with cycles rising from six *lakhs* cusecs to 15 *lakhs* cusecs and falling back to six *lakhs* cusecs above Naraj to observe the flow conditions with higher flood discharges. An interesting common feature of both these experiments was that the throw-off due to the spur increased with increased discharge. It was also observed that with higher floods, though both 1,400 feet and 1,200 feet long spurs worked satisfactorily as regards surface flow, yet the bed flow tended to approach the left bank below the spur much earlier

with the 1,200 feet spur than with the 1.400 feet long spur—a feature which indicated that with the 1,200 feet long spur the bed near the left bank would build up more quickly. The velocities near the right bank also increased causing a little scour of the extensive shoal near it.

SCOUR AT THE NOSE OF THE SPUR : DESIGN OF LAUNCHING APRON

In order to get quick results the model spurs described above were made of a wrought iron sheet having a slope of $\equiv 2\frac{1}{2}$ to 1 at the nose. The sides were vertical. The maximum scour depth obtained at the nose with this spur went down to R. L. 14. Having done these quick tests, a proper spur, with $\equiv$ 5 : 1 slope of the nose and representative side slopes, was constructed and the lowest scoured bed observed was only R. L. 28 (*i.e.*, maximum depth = 52 feet) which is approximately $2D$ Lacey as shown below :

$$D_{\text{Lacey}} = 0.473\ (Q/f)^{\frac{1}{3}}$$

where Q is the maximum discharge and f = the Lacey silt factor $= 1\cdot76\ \sqrt{m}$

In this case $Q = 4\cdot5 \times 10^5$ cusecs ;

$$\text{and } f = 1\cdot76\sqrt{1\cdot115} = 1\cdot86$$

$$\therefore D_{\text{Lacey}} = 0\cdot473 \times \frac{(4\cdot5\times10^5)^{\frac{1}{3}}}{1\cdot86} = 29\cdot47 \text{ feet}$$

$\therefore 2\,D_{\text{Lacey}} = 58\cdot94$ against observed depth in the scour pit $\equiv$ 52 feet. With scour equivalent to $2\,D_{\text{Lacey}}$ to be on the safe side, the maximum depth in the scour pit will be about 59 feet. If, therefore, a launching apron 29 feet wide $\times$ 4 feet thick is laid round the spur nose, it will launch at a natural angle of repose forming a slope of about 2 : 1 down to the bottom of the scour pit. This apron provides an allowance of 25 per cent. for stone likely to be washed away ; but some more stone was recommended to be kept in reserve for emergencies.

The nose of the spur and the first 10 feet length of the shank should be entirely of stone, the latter with a view to provide a good junction of the nose with the shank. The shank also will have to be protected by an apron 15 feet wide and 4 feet thick, both on its downstream side as well as on the upstream side.

The design of the spur is shown in Figure 2 Q.10. The nose of the spur is designed to slope down at 1 in 3 for top 1/3rd depth of flow, in order to prevent "gulleting", the remaining 2/3rds bottom portion sloping down at 1 in 5.

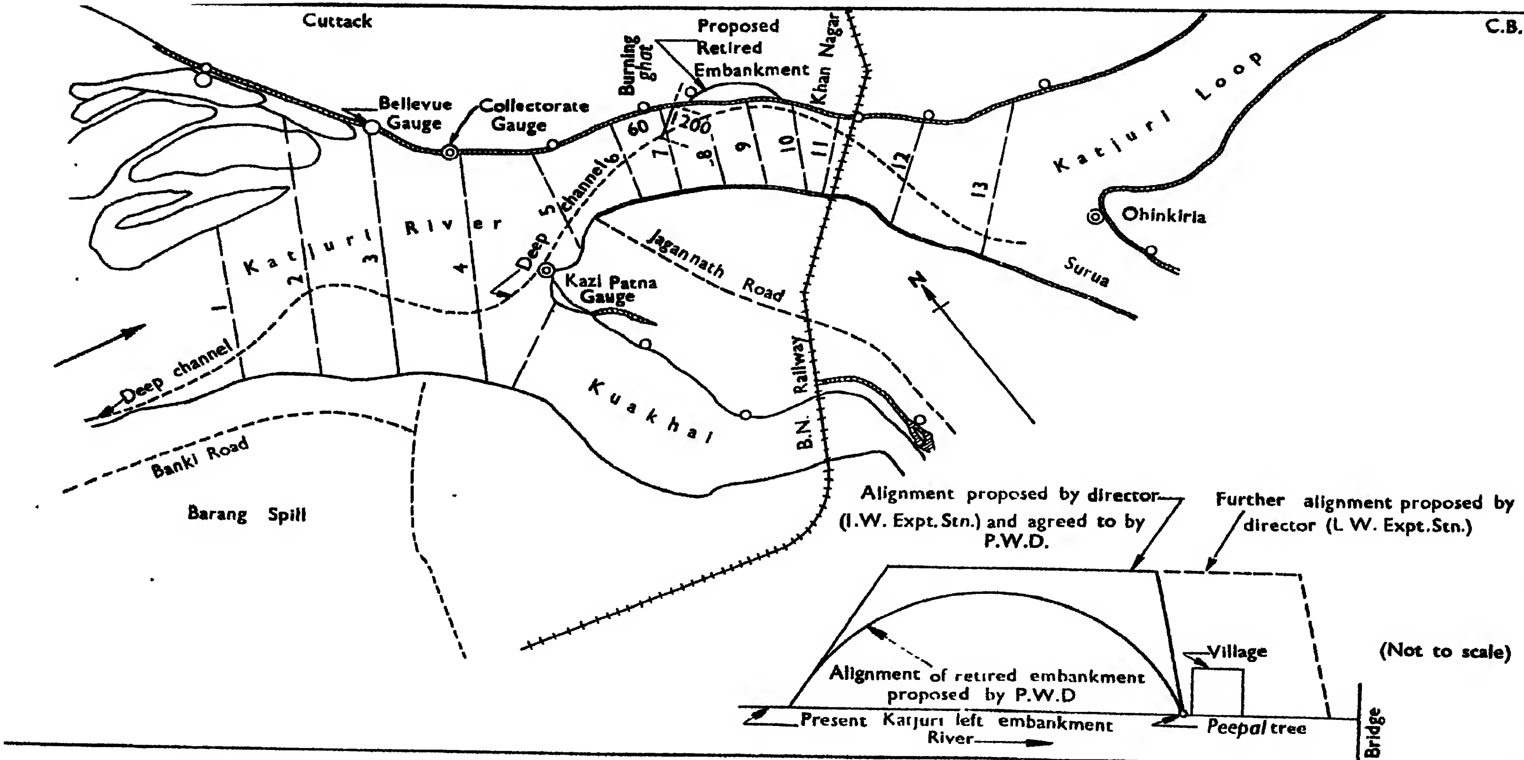

Figure2C.9:- *Showing the proposed 1,200 feet long repelling spur in the Katjuri River*

Scale 4,000 2,000 0 4,000 8,000 12,000 Feet

No. 2737 XDD'(C) 50 1,210 (P Z O)

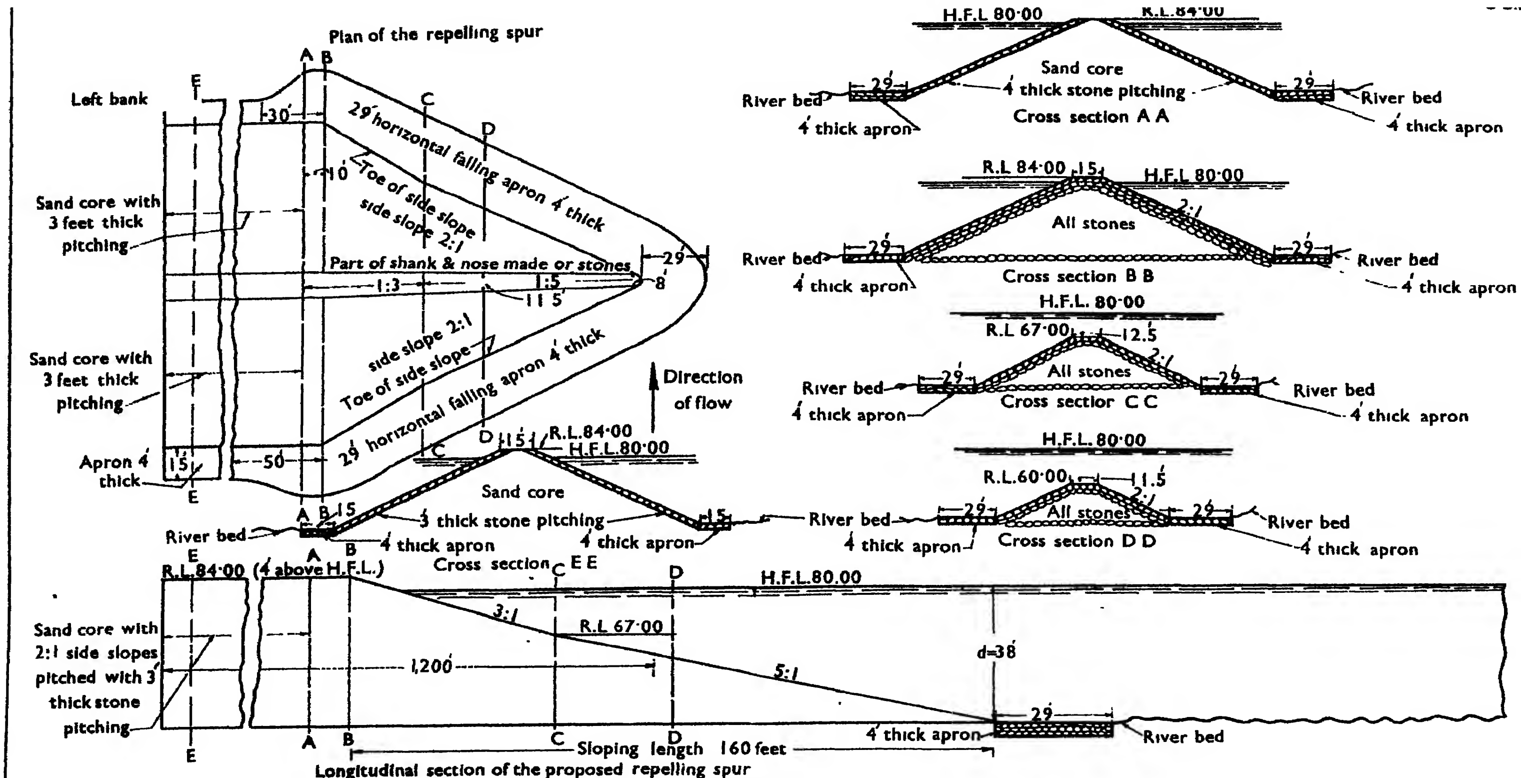

Figure 2C.10:- *Showing plan, longitudinal section & cross sections of the proposed repelling spur in the Katjuri River at Cuttack*

Reg. No. 2737 XDD (C) 58-1,250 (P.Z.O.)

CONCLUSIONS

The model does not give the rate and depth of accretion below the spur. It only gives an indication of the extent of accretion. This is because the bed changes in a model occur mostly on account of rolling bed sand ; but, in the river, it occurs more rapidly due to the enormous quantity of suspended load ; and hence the rate of accretion is very much slower in the model than in the river.

During the time the bed near Khannagar is building up to suit new conditions a certain amount of diving flow—certainly less severe than under existing conditions— may be anticipated and taken care of : but this will disappear as soon as the bed gets built up.

In the neighbourhood of the spur due to the restricted waterway, the extensive shoal on the right bank will scour and also the right bank may erode ; but this bank erosion is considered by the local officers to be of little consequence.

To counteract sufficiently the present severe conditions and avert the impending danger to the left embankment, the Khannagar revetment and the village, the optimum length of the spur should be 1,200 feet. However, later when the deep channel has shifted to midstream at Khannagar, after analysing the post-spur conditions. it may be possible to reduce with advantage the length of the 1,200 feet spur now recommended, if necessary for reducing action along the right bank.

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(6) TRAINING OF THE GANGA RIVER AT KANPUR[11]

Since 1900, the Municipal Board had been drawing water from the Ganga for the unfiltered water supply of Kanpur, situated on the south bank of the river. Up to 1912, the river course used to hug the south bank ; but since then it has been shifting northwards, necessitating the dredging of a channel to feed the pumps. During the last few years, the main current of the river has been flowing 1½ miles to the north of the Municipal Board's Pump House.

The unfavourable meander conditions of the river indicated that no permanent remedy could be evolved. After the 1944 floods the river conditions appear to have been improved and, accordingly, the measures suggested, without model experiments, consist of a pitched " attracting " island about 1½ miles upstream in the river and a pitched curved bank at the Pump House provided with an adequate launching apron.

The island was suggested, because a part island—called a *bund* when constructed in May 1947 by the Kanpur Municipal Board -had shown signs of improving the river curvature : and the pitched embankment followed from the analogy of the Yamuna at Delhi where a similar design was shown by model tests to be effective in "capturing" the river near the Delhi Gate Pumping Station.

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(11) Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report Technical, 1947, page 46.

(7) KSHIPRA RIVER AT UJJAIN—GWALIOR STATE[12]

ABSTRACT

Describes the preliminary history of the problem and the method of working out the model scales.

The city of Ujjain draws its water supply form an infiltration gallery situated in the bed of the adjoining Kshipra River. The infiltration gallery was constructed to a length of 200 feet in the years 1901-1903, and due to the increased demand of water, as a result of the tripling of the town population, the length of the gallery was increased to 600 feet. The gallery is constructed on rock foundations; it is rectangular in section (4 feet × 7 feet) with an arched *pucca* top. Dry stone masonry is built between side pillars spaced six feet apart to collect the filtrate from the sides. In order to have an assured depth of water over the infiltration gallery, throughout the cold and hot seasons, a barrage was constructed in 1931, half mile downstream of the gallery. The barrage impounds 14 feet of water by means of a double row of six inches high *kurries*, staunched on the upstream face with mud to prevent leakage, and has only two scouring sluices at present. During the floods only the top three feet of *kurries* can be removed, thus having a permanent ponding of 11 feet of water over the barrage sill. This, however, created a serious problem of accretion of fine silt and clay occurring upstream of the Barrage on falling floods. To get sufficient water, ponding is inevitable; while, because the infiltration head immediately after the floods, exceeds the critical head of infiltration the gallery has often got clogged with fine silt.

Further, the gallery being situated on the convex side at the apex of the meander, the river has been gradually receding away from it and eroding its concave right bank. This also called for protection of the right bank particularly of the historical observatory known as 'Jantar Mantar'; which was done by paving the side slopes with reinforced concrete slabs and pitching. To protect the right bank from scour and to push the current over the infiltration gallery, a series of small spurs were proposed by the local officers.

In consideration of all aspects of the case, the construction of a single, *attracting* spur No. 3 on the left bank opposite a point midway between spurs Nos. 7 and 8 on the right bank, Figure 2 C.11. It was, however, thought that this single spur may take a long time to act and may worsen conditions at 'Jantar Mantar.' So before the floods of 1944, repelling spurs No. 1 and 2 on the right bank as well as spur No. 3 on the left bank were constructed. This *attracting* spur No. 3 was, however, not given sufficient time or a fair chance to come into action because after the first low floods of August 1944, it was cut down by a length of 150 feet from its nose, while after the peak floods it was further cut down to 230 feet. Later, spurs 4, 5, 6 and 7 were constructed.

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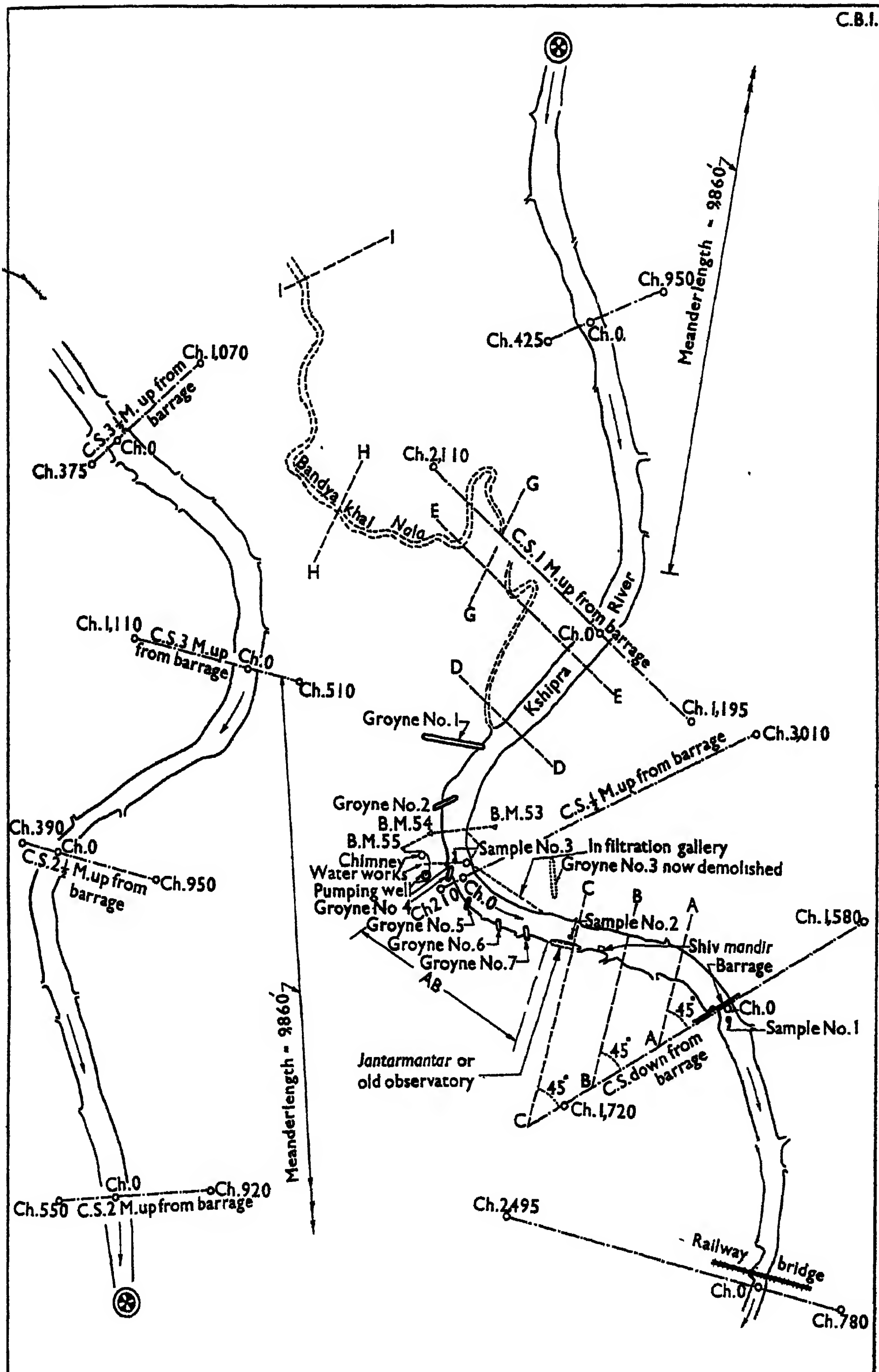
[12] Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report, Technical, 1947, pages 47-50.



Reg No 2737 XDD (C) '50-1,250 (P.Z.O).

Figure 2C.11:- *Showing survey plan of Kshipra River*

(3½ Miles upstream of barrage to ½ mile downstream of barrage)

Scale 500 250 0 500 1,000 1,500 2,000 Feet

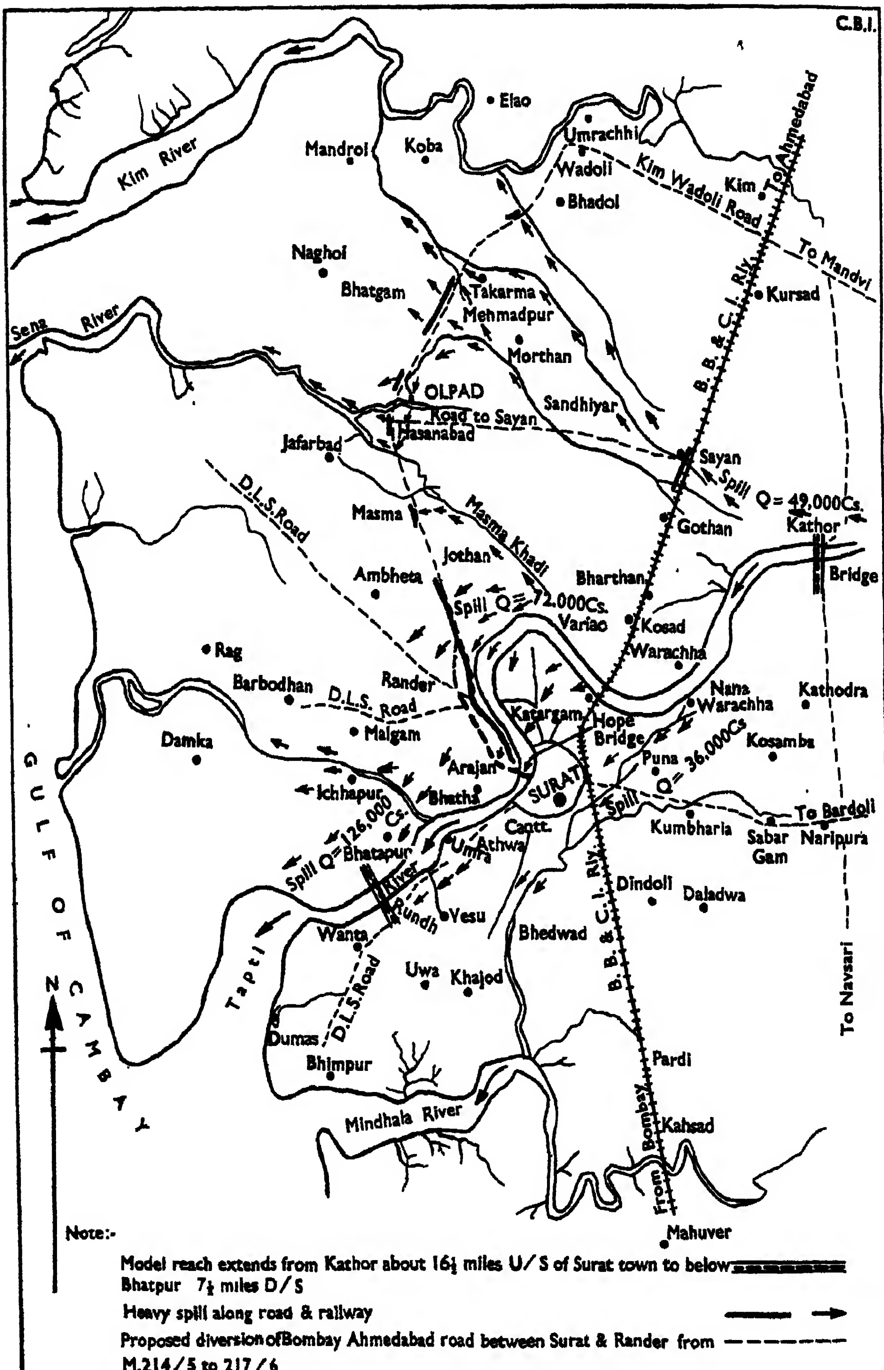


Figure 2C.12:-*Showing Tapti River near Surat during flood of August 1944*

Scale 4 2 0 4 8 Miles

Reg. No. 2737 XDD (C) '50-1,210 (P.Z.O.).

The Government of Gwalior State are now planning to pump raw water direct from the river. The site of this pumping station is to be on the right bank and the reach AB of the river has, therefore, to be protected from erosion. This scheme may take some time to materialize, and in the interim period the infiltration gallery has to be kept clear from getting clogged with fine silt.

With this twofold object in view, (*a*) of protecting the right bank, and (*b*) of preventing fine silt deposition on the infiltration gallery, a model of the Kshipra River has been recently constructed at this Station. The model extends 3½ miles upstream and ½ mile downstream of the Barrage. The model reach is shown in Figure 2 C. 10.

HYDRAULIC DATA

As in the majority of cases, the available hydraulic data is very meagre.

(*i*) *Bed sand*—Representative samples of bed sand collected from the bed near the infiltration gallery and a quarter mile upstream of the barrage were analysed and gave m=3·8 mm. and 5·75 mm. respectively.

(*ii*) *Bed and surface slopes*—The river bed slope, in the reach under investigation, is 1 in 2,500 (approx.) and the surface slope is 1 in 4,400.

(*iii*) *Maximum flood discharge*—There are no records of velocity and discharge observations. The maximum flood velocity, as estimated by eye by an officer, is reported to be 15 feet per sec., which appears much exaggerated, as will be seen from the discharge computation below. Similarly, the maximum flood discharge is reported to be 2·98 *lakhs* cusecs—presumably based on the overestimated flood velocity. This is a flashy type of river, and the floods rise and fall quickly.

COMPUTED ESTIMATES OF FLOOD DISCHARGE

From Figure 2C. 10 it will be seen that the meander length of the river is about 9,860 feet which gives,

from the Station formula $M_l = 30\sqrt{Q_{max}}$

$$Q_{max} = \left(\frac{9860}{30}\right)^2$$

$$= 1 \cdot 08 \text{ } lakhs \text{ cusecs };$$

whereas the meander length based on dominant discharge is given by the formula $M_l = 40\sqrt{Q\ \mathrm{dom}}$

from which Q_{dom} = 61,000 cusecs.

These figures are, however, based on only one meander length available

M21CBI

and, therefore, need corroboration.

Q_{max} from Lacey's general flow formula

$$V = 16 R^{2/3} \; S^{1/3}$$

R=15·05 feet and S=1 in 4,400 at the barrage section

$$V = 16 \times 15 \cdot 05^{2/3} \times \left(\frac{1}{4400} \right)^{\frac{1}{3}}$$

=5·95 feet per second.

It has been observed that the cross-section of the river 100 feet downstream of the railway bridge up to R. L. 1585·0 is quite stable and can be used for computing area of cross-section upto R.L. 1585·0 total area=$3 \cdot 47 \times 10^4$ square feet.

For areas which are overflow portions a coefficient value of 0·1 is assumed in working out effective velocity, while a coefficient value of 0·75 is assumed for the others and $Q_{max.}$=1·39 *lakhs* cusecs.

Similarly, with area of section up to R. L. 1575·5, *i.e.*, bankful stage,

Q_{dom}=0·70 *lakhs* cusecs

$Q_{max.}$ with the Mississippi formula :

$$V = [(3 \cdot 0966 \; R\sqrt{S})^{1/4} - 0 \cdot 0388]^2$$

where

$$R = \frac{\text{total area}}{\text{wetted perimeter} + \text{surface width}}$$

S=slope per mile

$$V = [\, 3 \cdot 0966 \left(\frac{5 \cdot 07 \times 10^4}{6120} \right) \times 1 \cdot 086 \,]^{\frac{1}{4}} - 0 \cdot 0388^2$$

= 5·10 feet per second

$$\therefore \; Q_{max} = A \times V$$

$= 2 \cdot 34 \times 10^4 \times 5 \cdot 10$

=2·19 *lakhs* cusecs.

CHOICE OF MODEL SCALES

(*i*) *Choice of length scale*—On considering the available and suitable space and discharge it is proposed to adopt a length scale of 1/170 giving

Q_{max} =1·39 *lakhs* cusecs≅4·81 cusecs in model,

and Q_{dom} =0·70 *lakhs*≅4·42 cusecs, in model.

(*ii*) *Choice of depth scale*—

(*a*) Applying Lacey's formula with m_m=0·5 mm. and m_p=3·78 mm.

$$\text{Vertical Exaggeration} = (Q_p/Q_m)^{1/6}\ (m_p/m_m)^{1/6}$$

$$=(170^2)^{1/6}\ (7\cdot56)^{1/6}$$

$$=5\cdot54\times1\cdot40$$

$$=7\cdot76$$

∴ *d* scale=1/21·9 or say 1/22.

(*b*) The vertical exaggeration was also worked out from the empirical formula (obtained from models of the range of discharge and size of the Kshipra model) :

$d=0\cdot68q^{0.71}$; which is suitable for conditions, where the bed charge is only moderate, as in the case of the Kshipra which is relatively narrow and deep, indicating low charge of bed material. The size of bed material was nearly the same as used in experiments from which the station formula was obtained.

w_p=900 feet and d_p=16·44 feet for dominant discharge at the Railway Bridge.

$$w_p/d_p=54\cdot7$$

$$w_m=5\cdot3 \text{ feet and } q_m=0\cdot775 \text{ cusecs}$$

$$\text{So } d=0\cdot68\ (0\cdot775)^{0.71}=0\cdot57 \text{ feet.}$$

$$\therefore\ w_m/d_m=9\cdot6$$

or vertical exaggeration$=\frac{54\cdot 6}{9\cdot 6}=5\cdot 67$ (as compared with vertical exaggeration according to Lacey formula$=7\cdot 76$).

$$d \text{ scale}=\frac{170}{5\cdot 67} \text{ or } \frac{1}{29\cdot 9} \text{ or say } \frac{1}{30}$$

A depth scale of $\frac{1}{30}$ was adopted as a first approximation.

(*iii*) *Slope scale ratio* : Again applying Lacey's formula,

$$\text{Slope Exaggeration}=(m_m/m_p)^{5/6}\ (Q_p/Q_m)^{1/6}$$

$$=(0\cdot 50/3\cdot 78)^{5/6}\ (170)^{1/3}$$

$$=0\cdot 185\times 5\cdot 55$$

$$=1\cdot 027$$

PROVING THE MODEL

According to Lacey figures, vertical exaggeration $=7\cdot 76$ and slope exaggeration$=1\cdot 027$; when the model was run for proving it, it was noted that it did not reproduce prototype conditions of flow correctly till it was given a bed slope exaggeration of $2\cdot 0$. Thus the model is tilted and has now vertical exaggeration$=5\cdot 67$ and bed slope exaggeration$=2\cdot 0$.

Apart from the opinions of experienced local officers (the Sanitary Engineer and the Executive Engineer) who have satisfied themselves that the model reproduction of existing conditions is in close conformity with the prototype, there is factual evidence of the regions of attack and high velocities and the flow pattern being the same in the model as in the prototype.

(8) MEASURES FOR ARRESTING BANK EROSION NEAR BHADELI AND BHAGDA VILLAGES ON AURANGA RIVER—BOMBAY(13)

ABSTRACT

Both the banks of the Auranga River near Bhadeli and Bhagada villages in Bulsar Taluka, Surat District, were heavily eroded due to combined action of the tides and floods. Investigations revealed that extensive measures for general training of the river would be prohibitive in cost and hence only local works designed to protect the important properties along the banks were possible. With this view, seven groynes on the left bank and three groynes on the right bank were constructed by local officers from time to time till 1941. The groynes on the Bulsar side were fairly successful in pegging the river to the left bank. The erosion, however, was moving downstream, as erosion generally does, and in doing this the natural swing of the river shifted to the opposite right bank. To check the consequent erosion, three groynes on the left and two groynes on the right bank downstream of the existing ones were proposed to be constructed ; views on these proposals are expressed.

(9) PROTECTIVE MEASURES FOR SEVERAL RAILWAY BRIDGES ON MADRAS-BOMBAY LINE OF MADRAS AND SOUTHERN MARATHA RAILWAY, BETWEEN MILES 82 AND 113(14)

ABSTRACT

A number of Bridges between Pudi and Urampadu—miles 82 to 113 on the Madras-Bombay line of Madras and Southern Maratha Railway, were severely damaged in the December floods of 1946, regarding which recommendations were made. The advice on points common to the design of several structures is given.

ESTIMATES OF MAXIMUM FLOOD DISCHARGES

The data obtainable in respect of flood discharges was inadequate. The estimates furnished, as based on catchment areas by Ryves' and Burges' empirical formulae, could not be taken as satisfactory as much depends on the correct assumptions of the widely-varying coefficients. For instance, Rhind's analysis has shown that C in Ryves' formula can vary from $C=308$ to $C=2707$(15). Figures obtained according to these formulae were significantly lower than those given by the Station formula duly corrected for shape of catchment. The Station formula is based on extensive data and checks up with several proven formulae—Ryves' formula with $C=2707$, in certain cases,

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(13) Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report, Technical, 1947, page 51.

(14) Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report, Technical 1947, pages 52-53.

(15) Buckley "Irrigation Pocket Book," Page 330.

gives about the same figures as by this formula. The recommendations were, therefore, based on discharges computed by the formula of normal floods as it does not appear necessary to provide waterway for the maximum—ever flood in view of the disadvantages following the formation of inerodible shoals and islands since the maximum—ever flood occurs but rarely.

BRIDGE WATERWAYS

The waterways proposed for the bridges had been calculated on certain assumptions of velocity which were not altogether warrantable.

The practice—pending the results of certain basic investigations in hand—is to design the bridge—waterway for the natural stable width of waterway ($P=C\sqrt{Q}$) for the conditions obtaining on the river. C has a value of 2·67 for Lacey conditions of " minimum charge with a fully active bed " and is modified in light of experience for divergence from regime—steep and shallow rivers indicating more bed load requiring wider waterways than flatter and deeper rivers. The recommendations were made accordingly.

DEPTH OF PIER FOUNDATIONS

Most of the bridges have open foundations carried to depths, which bear no relation to the conditions obtaining at the bridges.

The rational method of designing pier foundations—adopted in the Station recommendations—is to estimate the maximum probable scour and to allow for sufficient " grip " below the anticipated scour level.

Maximum probable scour round piers has been observed to be about 2 R (Lacey) ([16]) and a minimum grip of R is regarded as necessary to insure safety of the structure, where $R=0\cdot47\left[\frac{Q}{f}\right]^{\frac{1}{3}}$ and the Lacey factor f is (roughly) equal to $1\cdot76\sqrt{m}$, m being the sand grade in mms.

Data are still wanting to confirm the applicability of the Lacey formulae to boulder rivers and for their modification in cases of divergence from the implicit Lacey conditions of normal charge. In the meantime the large factor of safety usually adopted is inevitable.

([16]) Indian Waterways Experiment Station, Poona, Annual Report, Technical, 1944, Page 78.

HIGH LEVEL PAVEMENTS AND PITCHING ROUND PIERS

At several bridges bed pavements have been provided. Experience and experiments have both shown that high level pavements and pitching are not only unnecessary when sufficient grip is provided but are also undesirable, because they result in excessive scour below the bridge and round the piers(17).

It was, therefore, suggested that high level pavements and pitching should be avoided as far as possible, and if pitching is provided it should be laid at as low a level as constructionally feasible, being best placed at 85 per cent. of the maximum scour depth without piers(18).

CUT-AND EASE-WATERS

It was pointed out that the square—ended cut-and ease-waters, proposed to be adopted in some of the new piers—would cause appreciable obstruction to flow and increased afflux and turbulence. Right-angled, curved-noses, being economical and easier to construct and not significantly less efficient than the design with 'equilateral arcs of a circle', were recommended where flow expected to be axial; and, where flow might be skew, semi-circular cut- and ease-waters were suggested.

PROVISION OF ADEQUATE FREE-BOARD AND LOCATION OF BORROW PITS

A minimum free-board of two feet, above the maximum anticipated high flood level after making due allowance for accretion, was recommended to be invariably provided.

All borrow pits on the downstream side, particularly in the case of high banks, and borrow pits within 200 feet of the embankments on the upstream side, were suggested to be avoided. Sufficient ridges were advised to be kept between the borrow pits on the upstream side to avoid parallel flow along the toe of the embankment.

In addition to these general observations, advice was given in particular cases. In some cases, river training measures were suggested without model experiments and in one case model experiment was recommended. One case is described below :—

Bridge No. 251

The bridge being located immediately below an *S* curve, there was heavy attack on the left abutment and the end pier.

To relieve this attack a repelling spur was suggested.

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(17) Indian Waterways Experiment Station, Poona, Annual Report (Technical), 1939-40, pages 33-40 and 1941-42, page 12.

(18) Central Irrigation and Hydrodynamic Research Station, Poona, Annual Report, Technical, 1939-40, page 38.

Besides ameliorating the conditions at the bridge the proposed spur was designed to straighten the river upstream of it and in doing this it would cause a recession of the river away from the railway embankment in the embayment where it has approached dangerously close to the railway line.

In addition to the spur the following measures were recommended :

(*i*) Return walls round the north abutment.

(*ii*) Adequate strengthening of the collapsed right (South) guide bank and pitching with sufficient apron, in front as well as at the back ; the latter to afford protection against breaches from spills running parallel to the guide bank at its near slope.

(*iii*) Adequate waterway for the maximum discharge.

To avoid the inherent defects of the present structure, *viz.*, high pavement, uncertain shallow open foundations, inadequate waterway, unsatisfactory location below an *S* curve, the lack of suitable cut-and ease-waters *etc.*, and to overcome the doubts which must remain when a bridge is repaired at the same site where a pier has twice given serious trouble, it was proposed that the bridge should be relocated in the straight portion of the river.

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(10) TRAINING OF THE TAPTI RIVER AT SURAT (BOMBAY) [19]

ABSTRACT

The detailed history of the case is given in the 1944 Annual Report (Tech.) of the Station [20]. At Surat, the river Tapti takes a nearly right-angled turn and the curvature is too sudden and acute for the river to follow—especially in high floods. In the floods of 1937, the left abutment of the Hope bridge at Surat and the approach road to Rander were severely damaged.

Subsequently, in order to improve flow conditions and to accelerate erosion of the Central Island just upstream of the Hope bridge the Superintending Engineer, Northern Circle recommended blocking of the spill channel on the right bank behind the bridge abutment and raising of the loose stone floor in the left bank spans where it was deeper than the average bed level.

These proposals required to be contained for the reasons that the blocking of the spill channel would cause more afflux and raising the bed of the left bank spans would distort the natural shape of the bed and flow pattern at the curve and would thus permanently worsen conditions, which were already bad.

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[19] Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report Technical, 1947, pages 54-60.

[20] Indian Waterways Experiment Station, Poona, Annual Report, Technical, 1944, pages 47-90.

Inspite of this caution, however, costly embankment was constructed across the spill channel in 1939. According to Superintending Engineer, Northern Circle's orders, the top of the pitching round the bridge piers of the whole bridge was also maintained at the uniform level (R. L. 65.0) and the river bed in-between piers was pitched to R. L. 53·0.

These measures were, as anticipated, far from successful in preventing the damage in the subsequent floods. In 1944 it was recommended that the spill channel near the right bank of the bridge should be reopened by building three or four spans each of 100 feet, and the top of pitching round piers should be lowered or allowed to scour naturally from R. L. 65.0 to R. L. 53·0.

The spill channel has had to be ultimately reopened by removing the embankment and the practice of making good the pitching stone with a view to maintain the bed level uniformly at R. L. 65 has also been discontinued. This has eased conditions a bit ; but more relief from flood damage consequent upon these measures owing to reduction of afflux will be apparent only in the next high floods. (Since this has been done, there have been no high floods in the Tapti).

Model experiments were considered as essential to evolve the optimum protective measures for arresting the bank erosion from Rander Town to Hope bridge on the right bank and the severe attack on the Town Walls below the Hope bridge on the left bank.

Two other points have also to be investigated on the models :—

(1) At Bhatpur, in the bight on the right bank below Surat, a strip of land, about 2 furlongs wide, has been eroded—in a length of about 1/2 mile. Measures to prevent further erosion of these fertile and valuable lands are required.

(2) A new steam power station is proposed to be constructed at Umra near Surat. The Electrical Commissioner, Government of Bombay, sought advice regarding the correct location of the Pump House for the Power Station to avoid trouble due to erosion or sanding in future.

RIVER MODEL

The model extends from Kathor bridge down to sea end, a length of about 24 miles. It is laid in 0·4 mms. Theur sand, to a horizontal scale of 1/330 and vertical scale of 1/72 Figure 2 C. 12. To obtain satisfactory bed movement and reproduce flow conditions exactly as on the prototype, a tilted model with Vertical exaggeration=4·58 and Slope exaggeration about 2·5 has been adopted, after preliminary trial runs.

The river being of the incised type, with meander imposed and maintained by the local *kankar* and hard clay formations, it was found that to simulate gauges the discharge scales required is $L_d\sqrt{d}$=20,200. The bed width to depth

ratio is small for the discharge, indicating little sand charge moving on bed. This also justified the adoption of discharge scale=$L_d\sqrt{d}$.

Time scale based on the assumption that tractive force in the model is equivalent to that in the river worked out to 1/1118.

Flow lines near Hope bridge at low stages of the river—at water level 80·00 (when the island above Hope bridge was just wholly submerged) and at water level 70·25 (when it was partially exposed)—were visually observed in the river.

In the model, after stabilisation, the flow lines were observed at water level equivalent to R. L. 80·00 ; but at water level 70·25 the depth of water was so low and flow so little that observations could not be done.

In comparing flow lines in the river with those in the model, it is to be remembered that ;

(1) observations in the river could only be made from the bridge and near the sides and, in between these sections, the flow lines have been drawn in by judgment, and

(2) in the narrow section below the bridge the river is so deep that the vertical exaggeration model would require side slopes steeper than the natural angle of repose of the bed material ; and with natural side slopes a relatively wider section is obtained at the bottom in the model. The concentration of flow occurring along the left bank in the river is, consequently, not reproduced to the same extent in the model. This is one of the model limitations which has to be given due consideration while interpreting results obtained with training measures.

When due allowance is made for these factors, it will be apparent that there is close conformity of flow lines in the model and prototype.

Flow lines starting from Rander and down to Bhatpur end, and velocities at typical sections, with the dominant discharge of 2·48 cusecs≡ 5 *lakh* cusecs, were also observed.

The high velocity flow closely hugs the bank throughout the Rander bend and the Bhatpur bight. This is evidently the root cause of the bank erosion at Rander and Bhatpur.

It was also seen in the model that, below Hope bridge, the flow squarely attacks the left bank between Makhi Khadi and Nanpura Post Office and undermines the buildings abutting against the bank due to diving flow.

At the proposed location of the Pump House near Umra, on the left bank about 2¼ miles downstream of Surat, under existing conditions the model indicated that a perennial deep water channel would not be ensured, the location being not sufficiently upstream of the transition of the main current from the left to the right bank.

Exploratory tests were carried out to devise suitable training works to solve the problems mentioned above. All these problems being inter-linked, in the sense that the consequences of one will be felt by the next, it is decided that the desired results should be obtained by a combination of a minimum number of mutually beneficial training works. Measures suggested by these exploratory tests for more detailed examination on the model were :—

(1) a repelling groyne on the right bank near Rander;

(2) a similar groyne on the left bank or a pitched island in mid-river somewhere between Makhi Khadi, and District Judge's bungalow ; the same measure will, if possible, be made to peg the deep water channel at the proposed site of Pumping Station ; and

(3) a pitched island or other training works for protecting Bhatpur bank.

Alternative training works *e.g.*, a pitched island upstream of the bridge did not hold out the same degree of promise.

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(11) MUSAPET ANICUT SYSTEM [21]

ABSTRACT

Two seasonal rivers, Kapeta Vagoo and Polkampalli Vagoo, meet about five miles from Mahbubnagar, a District Headquarters in the State situated on the Hyderabad-Bangalore Road. As is the case with the majority of small rivers in the Deccan, these rivers flow only during the monsoon season and a heavy downpour during this season causes high floods in the rivers. In summer they are entirely dry.

As excavations at the site of the anicut of Kapeta Vagoo have disclosed that proper foundations are not available even at a depth of 25 feet below bed level,

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[21] Hyderabad Engineering Research Laboratories, Annual Report, 1947, pages 15—17.

the construction of a new weir would be very expensive. If some river training works were to be put on after model experiments so that less sand is drawn during floods into the Polkampalli Vagoo, more sand will be deposited in the bed of the Kapeta Vagoo during the subsiding floods of each monsoon. Thus after two or three monsoons the bed level in Kapeta Vagoo will become higher than that of Polkampalli Vagoo thus allowing more water to be drawn into the Polkampalli Vagoo without having to construct a costly weir.

Model-Experiments:

A pilot model of the rivers was laid in the sand tray inside the Laboratory to the following scales.

$$\text{Horizontal} = \frac{1}{240}$$

$$\text{Vertical} = \frac{1}{36}$$

$$\text{Slope exaggeration} = 3 \cdot 33$$

As the sides and the bed were moulded of locally available sand screened to the required grade it was found that the model got eroded badly even when it was run at half the maximum flood level, thus indicating that the slope exaggeration was too much. The slope exaggeration was then reduced to 1·67 and the model was run to half the maximum flood level and it was found that the sides withstood erosion. The model was then run at this level for various runs and the movement of bed silt studied. As there were no results of previous surveys available the model could not be proved in the strict sense by studying the increase and decrease in bed level of each river after each run and comparing it with the prototype. But it was found that after each run the bed of the Polkampalli Vagoo progressively silted up and the bed of the Kapeta Vagoo eroded, and when a slight amount of fine sand was fed into both the rivers there was greater movement of sand into the Polkampalli Vagoo than into the Kapeta Vagoo. Thus it was taken that to a very large extent the model fulfils the conditions existing in the prototype.

In order to get an indication of the methods to be tried to encourage the silting up of the Kapeta Vagoo and discourage the transfer of bed sand into the Polkampalli Vagoo the following devices were tried.

(*i*) It was felt that the large island, marked A in the plan Figure 2 C. 13 with its sharp pointed nose upstream was probably responsible for more bed-load being drawn into the Kapeta Vagoo. Hence various trial runs were taken on

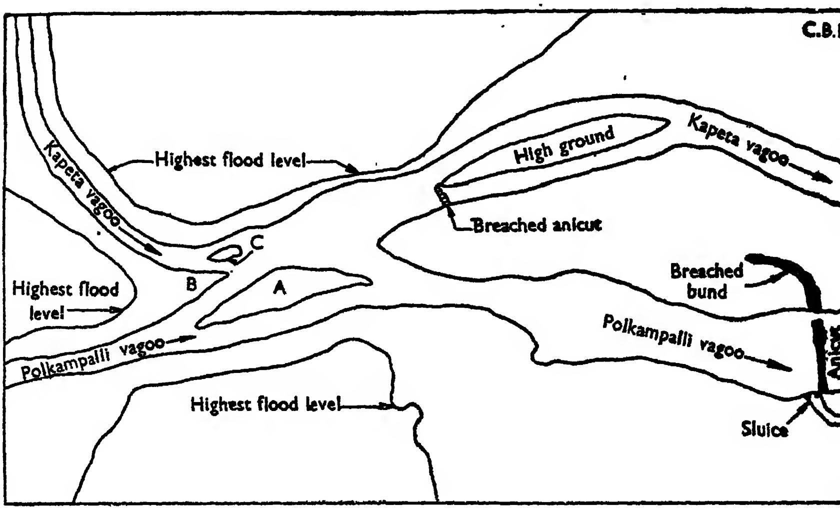

Figure 2 C. 13: *Survey plan of Musapet Headworks*

the pilot model by progressively cutting off the upstream portion of the island in the model. With each condition the model was fed by screened sand and it was found that with each progressive shortening of the island from the upstream side, more sand was drawn into the Kapeta Vagoo and less into the Polkampalli Vagoo. It was noticed at the same time that there was erosion just downstream of the nose (marked B in the plan). There was progressive deposition towards the left bank of Kapeta Vagoo.

(*ii*) As it was felt that the nose marked B may be responsible for the non-uniform distribution of sand in the bed of the Kapeta Vagoo, this was also progressively cut off and the model run. It was found that this improved the deposition of sand on the bed of the Kapeta Vagoo, and also reduced to a slight extent the inflow of sand into the Polkampalli Vagoo.

(*iii*) Next the effect of completely removing the small island (marked C in the plan) was tried. This resulted in a more uniform settlement of sand in the bed of the Kapeta Vagoo but had no noticeable effect in the inflow of sand into Polkampalli Vagoo.

(*iv*) Next the effect of providing spurs projecting from the left bank of the Kapeta Vagoo was tried. It was found that while these helped in the deposition of the sand near the bank, the sand in the centre of the river was eroded.

The above experiments were only of an exploratory nature because of the small scale of the model. On the basis of these preliminary observations a large scale model will be laid out and the various devices tried. This model will be laid out with inerodible banks and sand bed so that the model could be run upto high flood level.

DISCUSSION BY THE RESEARCH COMMITTEE

Introducing items (1) and (2) MR. S. N. GUPTA said that the Ganga river at Hardwar, had been getting unmanageable for the last few years. Below Mayapur, it took a turn towards its left bank leaving a spill towards Kankhal which threatened the very existence of Kankhal town.

Again, at Bhimgoda weir, the undersluices on the right were taking 53 per cent. of the discharge whereas the main weir was taking only 47 per cent. of a total flood discharge of 150,000 cusecs. This happened because the bays of the weir on the left were blocked and the supplies were diverted to the right to feed the New Supply Channel. This resulted in excessive velocities at the undersluices which in turn created retrogression downstream.

Model investigations were taken up to find remedial measures. The river bed consisting of boulder, shingle and sand, presented many difficulties in the proper selection of scales.

Eight miles of the Ganga river *viz.*, 3½ miles upstream of Bhimgoda weir and 4½ miles downstream of it had been reproduced in a vertically exaggerated model to scales 1/120 horizontal, and 1/40 vertical. The bed was laid with a mixture consisting of 25 per cent. of sand 25 per cent. of ¼ inch and 50 per cent. of ¼ inch to 1 inch shingle. These reproduced the required prototype conditions with a discharge scale of 1/22,000.

A 960 feet long boulder spur built at an angle of 120 degrees from chainage 1,400 of the existing Kankhal *bund* had successfully diverted the Ganga along its natural course towards the left ; as a result the spill channel below it started shoaling up.

The blocked crest of the Bhimgoda weir was lowered in Bays Nos. 3, 4, 5 and 6 and this ensured an equitable distribution of supplies *viz.*, 67 per cent. over the weir and 33 per cent. through the undersluices. It was hoped that this distribution would relieve the undersluices of the excessive strain which in turn, would reduce retrogression downstream without affecting the regime of the river upstream.

DR. N K. BOSE in introducing item (3) said that these investigations were in continuation of the work done last year. He had nothing to add to what was given in the report.

Introducing item (4) MR. S. T. GHOTANKAR said that as a result of model experiments a 1,270 feet pitched guide bank normal to the bridge and joined to the railway embankment by a 1,000 feet radius curve gave satisfactory conditions of flow at the bridge. Total length of guide bank including curved-head was 2,270 feet.

As regards item (5) MR. S. T. GHOTANKAR observed that experiments were carried out in the 1/400 : 1/66 vertically-exaggerated Mahanadi model. Various lengths and positions of the spur were tested and a 1,200 feet long spur pointing 60° upstream of the Burning Ghat—almost the same as suggested at site—was found to give optimum results.

RAO BAHADUR D. V. JOGLEKAR introduced items (6) to (10) and gave a brief account of each. Regarding item (9), he stated that a number of bridges between Pudi and Urampadu on the Madras—Bombay line of M. and S. M. Railway were severely damaged in December floods of 1946. Advice of the Director was sought on the remodelling to be done. His recommendations on points common to the design of several structures include :

(1) In absence of observed flood discharge data and in view of the unreliability of the empirical formulae like Ryves', wherein much depended on the correct assumption of the co-efficient, station recommendations were based on discharges computed by the proven Station formula of normal floods

(2) For the necessary bridge widths, natural stable widths of the water way for the conditions obtaining on the particular rivers were estimated.

(3) Most of the bridges had open foundations carried to depths which has little relation to the conditions obtaining at the bridge. Maximum scour round piers in the Station proposals was estimated at $2R$, where $R=0\cdot47\ (Q/f)^{1/3}$; and a further grip of R below this anticipated scour was suggested.

(4) High level pavements and pitching were suggested to be avoided.

(5) All piers to be newly constructed were recommended to be built with right-angled curved noses, being economical and easier to construct and not significantly less efficient than the design with 'equilateral arcs of a circle, where flow was expected to be axial ; and where flow might be skew, semi-circular cut-and ease-waters were suggested.

(6) A minimum free board of two feet above the maximum anticipated H. F. L. was recommended.

(7) All borrow pits on the downstream side, and within 200 feet of the embankment on the upstream side were suggested to be avoided.

In addition to these general observations, specific advice was given in particular cases. In some cases, river training measures were suggested without model experiments ; and in one case model experiments were recommended.

Referring to item (10), RAO BAHADUR D. V. JOGLEKAR drew attention to the history of the damage caused by the Tapti river in the vicinity of Surat since 1937 and the remedial measures tried with little success, as given in the Report.

The Station was ultimately approached for advice regarding the appropriate measures for arresting the bank erosion at Rander and Bhatpur, the undermining of foundations of buildings on the Surat bank below the Hope bridge and the suitability of the site proposed for the Pump House of the proposed thermal electric Power Station.

A river model of the reach from Kathor to the sea was constructed at the Poona Station to evolve and test the necessary measures. There was satisfactory bed movement in the model and flow lines observed in the river, when compared with those in the model, showed close conformity between the model and prototype.

The model indicated that the bank erosion at Rander and Bhatpur was due to high velocity flow hugging the bank ; while undermining on Surat bank below the bridge was due mainly to a direct attack by the main flow diving at the bank. The proposed location of the Pump House near Umra was found to be in the transition reach where the main flow crosses over from the left to the right bank and thus somewhat downstream of the suitable position.

Exploratory tests indicated the following tentative training works :—

(1) A repelling groyne at Rander.

(2) A similar groyne or island in mid-river somewhere between Makhi, Khadi and the District Judge's Bungalow, below the Hope bridge.

(3) A pitched island or other training works for protecting Bhatpur bank.

Detailed experiments were in progress.

As there was no representation from Hyderabad present at the meeting, item (11) was taken as introduced.

In the discussions that followed, MR. KUTTIAMMU said that there seemed to be some discrepancy in the flood levels noted in item (1) and (2). " May be, it was a printer's error "

He further pointed out that the model combined a rigid weir and a mobile river bed. A distorted scale had been adopted. He enquired whether any special precaution had been taken in reproducing the afflux at the weir. Also, what were the arrangements or devices made in the model for measuring the disharges through the different bays separately.

Mr. Govinda Rao referring to item (4) said that the guide bank to prevent erosion at the upstream end of the pitched bank as proposed now was to be 4,500 feet of the total length. This gave a bell mouth conditions of entry to the bridge on the right bank and made the condition of entry unsymmetrical and flow non-uniform. Due to the presence of a convex curve at the left bank and the pitching effect of the proposed right bank, the flow had naturally to get attracted towards the right bank. It had also been stated that a deep channel was formed along the toe of the embankment and during low floods there was scour along the toe of the guide bank.

Mr. Govinda Rao observed that there seemed to be a small island being formed on the left bank just beyond the nose of the spur. If this was pitched and if necessary, its nose properly oriented, he hoped, it might help to attract the flow into the middle of the river during periods of low floods.

He opined that the solution of the problems was closely interrelated with the problem of re-distribution of sand load between the Mahanadi and Katjuri Systems. It was worthwhile considering whether the spur suggested would still be necessary to protect the Khannagar bank in view of the Katjuri embankment developing a channel along the Cuttack bank. Regarding the experiments, after the accumulation of silt behind the 1,200 long spur, and after the Katjuri has assumed its regime, one would like to know what would be the distribution of sand load between the Katjuri and the Kuakhai. What would be the effect of silting up behind the spur on the railway embankment proposed on the right bank of the Kuakhai bridge ? Would this encourage further accumulation of sand at the convex *bund* between cross-sections 2 and 3 marked ?

By having a spur, the waterway got reduced and it was quite possible that more and more of the flow will get diverted towards the right bank. This might increase the discharge at Surua, and might cause spill or scour at Surua.

Another point that was worth noticing, was that there was a shoal formation at the nose near Khasipatna just where the river bifurcates into Katjuri and Kuakhai. The conditions of flood discharges and lines of flow promoting the growth of shoal and the consequence of its presence on further erosion might be worth investigating.

Regarding experiments done by the Poona Station on the protection of right bank of Kuakhai river above the B. N. Railway bridge Mr. Ghotankar remarked that the question was a peculiar one. If one started setting right a trouble at one place, the other side was affected. The station report explained the remedies proposed. Apart from the cost involved in the proposal due to unsymmetrical entrance there were other complexities. There should be greater compensation of flow. If a pitched island was formed in the middle, say between cross-sections 2 and 3 it might help to attract flow into the middle of the river during low floods. But it would cause danger on the cancave side.

He admitted that in item 5 the solution was closely inter-related with the re-distribution of the sand load between the Mahanadi and the Katjuri.

MR. R. R. HANDA said that regarding the Katjuri spur the work had since been completed. There was 14 feet depth of water on the top of the apron when he left Cuttack for Simla. The river was still attacking opposite Khannagar. There was a small island between the railway bridge and the spur (which was not shown in the report) in which he had ordered a cut to be made. The two works were independent. It was too early yet to say whether the work was successful or not.

Regarding the experiments at Poona, Mr. KHANGAR liked to know about the effectiveness of the pitched islands and attracting spurs. In the Punjab pitched islands had not been successful. The pitched island upstream of Suleimanki weir had been a complete failure. He wanted to know what had been the experience elsewhere.

He further asked as to what had been the experience of the Poona Institute regarding the resuscitation of dead rivers. The problem in Bihar was mainly one of resuscitation of dead and dying rivers. Most of the irrigation channels got choaked up and started dying off. If spurs were put in, they were likely to develop the trouble still more. Could the Poona station suggest some ways and means ?

MR. M. P. MATHRANI said that in connection with the model for the Mor river he would like to give a note of warning. The rivers of Bengal were of a special nature. The average discharges of these rivers differed very much from the ordinary discharges. Another thing prevalent in these rivers was the varying hydraulic conditions. Supposing there were no changes normally, the changes would appear after the construction of the dams. In all these rivers one had to consider the question of accretion on the upstream side. But from his experience that had been gained in Sind, the accretion problem on the upstream side was a problem which started creating considerable trouble. He thought that in the rivers of Bengal this problem was almost the same. For the construction of barrages of dams in these rivers, the hydraulic conditions of the rivers should be borne in mind.

RAO BAHADUR D. V. JOGLEKAR referred to item (1) and said that the Kankhal spur as proposed would attract the flow of the main channel and would be subject to a very severe attack on the whole of its upstream face in course of time.

What was really required in this case was a repelling spur facing upstream.

The slope of the nose proposed was 1 in 5. The Station experience with such flat slopes was that gulleting occurred *i.e.*, stone pitching was washed away due to local high velocities to about 1/3rd the depth from water surface and in such cases it was better to give 1 in 3 slope up to this point and then have a flat nose of 1 in 5 lower down.

Regarding item (2) remarked that for equalizing flow, the crest of left bank spans were lowered to increase the discharge intensity and experiments showed that the discharge was equally distributed for $Q=153,000$ cusecs. Actually the maximum discharge was stated to be 670,000 cusecs. Surely the conditions would materially alter with this discharge and it appeared almost certain that the high intensity would persist along the right bank in spite of the lowering of the sills in the left bank spans. One could never equalize the flow in this way unless the curvature of flow was suitably altered upstream. This was made clear in the case of the Tapti River at Hope Bridge in which an attempt was made to equalize the flow by raising the deep portion of the river near the left bank spans with stone pitching. This completely failed and conditions were made worse.

With regard to Mor Barrage Experiments RAO BAHADUR D.V. JOGLEKAR had the following comments to make—The crest level of the undersluices =R. L. 192 and that of the Head Regulator=R. L. 199. Why was it necessary to have the sill of the Head Regulator so high? It was a misguided believe that when the sills were high bed sand would be excluded. The main factor that controlled sand was the correct curvature of flow. If the curvature was wrong sand was drawn into the canal in spite of the sill of the regulator being high. This had been proved so many times in all their model experiments and in prototype.

Similarly the spurs which were stated to have worked statisfactorily in the model were short spurs and would peg the flow on the Right Bank on which they would be constructed. The short spurs proposed in this case were too near the barrage and might create turbulence which would adversely affect the Right Bank Canal.

With the proposed barrage site at cross-section 2-A it appeared extremely doubtful whether satisfactory conditions for Left Bank Canal could ever be obtained. On cross section 3-A, a central pitched island which had given extremely satisfactory results in the case of the Lower Sind Double Barrage model at Kotri might be tested. This might ensure favourable curvature of flow on both banks.

MR. R. R. HANDA said that he took over charge in Orissa at a very late stage. At that time it was too early to change the design of the spur because this would have needed performance of experiments. This could be risked because delay would have been risky for Cuttack.

He found that the stone for the pitching of the Katjuri spur had to be carried over a long distance (12 miles). This was being carried from an ancient monument. The Archaeological Department objected to it but they managed to evade that Department, in interest of completing the work. As, however, there was difficulty in getting the necessary quantity of stone, the thickness of the side slope pitching had to be reduced from four feet to two feet.

If instead of this solid stone nose spur a "T" headed spur was constructed it would have cost less. At Suleimanki headworks, now in the West Punjab, he had constructed solid stone nose spurs as well as "T" headed spurs and he had found the latter far more economical than the former.

Dr. N. K. Bose remarked that repelling spurs would be dangerous in an alluvial country as one did not know how far the repulsion would act. He added that near Khannagar, the level of the country was very undulating as was peculiar to Bihar and Orissa. If a channel was dug somewhere the river might change its course and go off several miles. He wished the model had been run for a longer duration. The sand samples too were curious. They seemed to vary at every quarter of mile.

Mr. S. T. Ghotankar speaking on Mor Barrage model said that it had been stated "If sand is used in the model as the silt entering the canals for purposes of model study, consistent results are not obtained in a mobile bed river model." The experiments reported herein appeared to have been done on an erodible model so he would like to know whether these results lay on the consistent side or the inconsistent side. The difficulty about sand entering the canals at the time of initially letting in the discharge could very easily be avoided by letting in discharge from the downstream end and gradually ponding and soaking the model bed.

Secondly, it was stated in the report "The river model was first built on a horizontal scale 1/200 and vertical 1/80. These scales were chosen because they were found to work well in the case of the river model of the Damodar." In this case the discharge of the Damodar had been stated to be 6·25 *lakhs* cusecs whereas that of the Mor Barrage was two *lakhs* cusecs. If this was the case how could the same scales be adopted for both the river models?

Mr. C. V. Gole remarked that the approach to the selection of the suitable site for the Mor Barrage left much to be desired. Firstly no mention was made of experiments without a barrage to indicate flow under existing conditions. Merely deciding the position of the barrage on consideration of the extent of surface return flow was not enough.

Mr. S. N. Gupta said in reply to Mr. Kuttiammu that they had taken sufficient precautions in their model work. They tried to maintain proper gauges at different parts of the model by injecting sand and shingle on the upstream side of the weir. As to discharges they observed velocities in the model by pilot tube and in the prototype by surface floats and found that the discharges were varying only by five to ten per cent.

Regarding Rao Bahadur Joglekar's question he admitted that a slope of 1 to 5 was in view for the nose. His further suggestions in this connection were welcome. The sluices at Bhimgoda were designed for 150,000 cusecs and the lowering of the crest in the left portion of the weir allowed better distribution of flow across the weir. The maximum discharge occurred for a period of 1½ months during the monsoon. The authorities were mainly afraid that the whole river might not go to the left bank and this might not deprive the New Supply Channel of its discharge. Thus they were left with no other course for them tor training the river.

Rao Bahadur D.V. Joglekar replying to Mr. Govinda Rao said that these two proposals were independent of each other. The flow into the Kuakhai was from left bank at the head to the right bank at the bridge and it was impossible to train the river in this short reach and hence one had to accept the situation of a high velocity flow along the right bank and must provide for sufficient apron protection as already recommended.

As regards the Khannagar spur on the left bank of the Katjuri, the main purpose was to protect the Cuttack town which is situated on that bank. There was nothing very important on the right bank and hence it did not matter if the right bank was scoured away. The railway bridge was also so wide that it could easily take additional discharge along the right bank. The Surua river was so far downstream that it would not be affected by the spur.

Rao Bahadur D. V. Joglekar replying to Rai Bahadur S. D. Khangar said that it was the first time that he heard of the islands not being successful in the Punjab and in the particular the Suleimanki Island. The Punjab Research Institute had been advocating during the last three or four years that islands should be used wherever possible and their model experiments had also shown islands to be very effective. On the other hand his Station had always stated that the results obtained by the Punjab were not correct and that islands should not have been recommended in those cases. As a matter of fact the Station held the view that islands or spurs could be found suitable in individual cases according to the conditions obtainable and they could not lay down any universal rule about their efficacy. So far the Station had not recommended any island except at Kanpur, where they thought it would be useful. An example of an attracting spur could be seen in the case of spur No. 5 on the Yamuna at Delhi downstream of the Railway Bridge. It had pegged the river permanently at its nose.

Fortunately the nose was outflanked in the last year's flood and the Station had recommended that it should not be restored as it was excepted that the river would break away from that bank and would be attracted by the stone pitched embankment proposed near the Delhi Gate Pumping Station on the right bank.

As regards the revival of dead rivers, it was very difficult to give a general opinion and each case would have to be dealt with on its merit.

RAO BAHADUR D. V. JOGLEKAR next said that the difficulty mentioned by Dr. Bose had never been experienced at the Poona Station. The procedure followed the Poona Station was that the model was run to stable conditions after running four or five flood cycles and the following observations were done by running the dominant discharge :—

(a) Lines of flow at surface and bed which indicated qualitatively the amount of bed sand drawn by the canals;

(b) A certain amount of sand was injected and the amount of sand collected on the bed of canals was measured in certain time; and

(c) Seeds were dropped on the bed of the model and the number of seeds collected on the bed of canals measured and percentages worked out.

These three sets of observations gave an idea of the amount of sand that would be drawn by the canals in the prototype qualitatively.

DR. N. K. BOSE asked how the samples of the Kshipra at Ujjain were taken? The grades varied considerably.

RAO BAHADUR D. V. JOGLEKAR replied that this was an "incised" river flowing through very uneven country and the variation in the grades was due to the rapid changes in the flow conditions. The samples could not be taken during floods as the depth of water was great; hence these were taken after the floods had subsided.

DR. N. K. BOSE enquired as to what was meant by an "incised" river? Did the meander formula hold good for the incised river?

RAO BAHADUR D. V. JOGLEKAR replied that an incised river was one which cut its bed in contrast with rivers in flood plains, which build up their beds. An examination of meanders in both types of rivers carried out at the Station had revealed that meanders were a function of discharge. This had already been described in their previous Annual Reports.

DR. N. K. BOSE in reply to Mr. Joglekar said that pitched island was tried on the Mor Barrage model but the normal discharge of the river was very low and it did not come into operation. The discharges at which this island was effective lasted only for a very short period and as such, the influence of the island could not be felt.

Different types of spurs were tried on the model. It was found that the downstream spur as proposed in the model protected the bank and did not produce any turbulence so that the silt entry in the right pocket was not increased. Dr. Bose said that the model was run without the barrage. The result of this run was described in the report of 1946.

DR. BOSE said that the crest of the canal regulator was raised above the floor of the undersluice. This was effective in excluding very coarse silt of which a considerable quantity was carried by the river.

He added that the scales in the Damodar and Mor barrage models were kept the same, as only one kind of sand was available for both.

Dr. H. L. Uppal wished to speak a few words about pitched islands. Regarding the pitched island that had been used on the Suleimanki headworks, it was situated about 450 feet upstream of the Suleimanki weir and was associated with a specific regulation, which was given up in 1946. This was the reason why it failed to come into action. But that any service could fail a year or so *e.g.*, the training works at Madhopur in 1947 and this did not prove that the pitched island was always ineffective.

DISCUSSION BY THE BOARD

The Secretary said that 11 items were discussed at the Research Committe meeting (page 614). There was no resolution. As regards the proposed C. B. I. Publication on 'River Training' the position was the same as stated in the Preliminary Note on page 613.

It was decided to retain the subject on the Agenda.

(iii) SCOUR AND EROSION

PRELIMINARY NOTE

The Central Waterways, Irrigation and Navigation Research Station is collecting data on scour below bridges and it is hoped that the results will be presented to the Research Committee as soon as they are available.

The following items were discussed at the 1947 Research Committee meeting :—

(1) Training of the river Indus above the Sukkur Barrage to prevent slips of the right bank below the nose of the approach channel.

(2) Emergent measures for preventing serious damage to the central sluices for the Mahanadi Anicut at Cuttack, Orissa.

(3) Scour downstream of rigid structures in wide channels—prototype data regarding depth of scour downstream of bridges.

(4) Coffer dam for the Ramapadasagar Project.

(5) The Hagari Aqueduct for the Tungabhadra Canal—Scour around piers.

Recent Literature.

(1) Remodelling the Esna Barrage—Water and Water Engineering, Vol. 50, No. 613, March 1946.

(2) The Bulletin of the Beach Erosion Board—War Department, Corps of Engineers, U. S. Army Office of the Chief Engineers, Washington, D. C., Vol. 1, No. 1, April 1947.

(3) Haas R. H.—Development of concrete revetments on the Lower Mississippi—Parts I and II Concrete, Vol. 55., No. 4 and 5, April and May 1947.

(4) Gardner W. and Lawritzen C. W.—Erosion as a function of the size of the irrigation steam and the slope of the eroding surface—Soil-Science, Vol. 62, No. 3, September 1946.

(5) Framji, K. K., Director Central Waterways, Irrigation and Navigation Research Station, Poona—Scour below weirs—Second meeting of the International Association for Hydraulic Structures Research. Stockholm. 1948, Paper No. 3.

(6) Tisonl. J., Professeur a l'iluiversite de Gand, Directeur de laboratorie d'hydraulique.—Transport de materiaux de fond, et Erosion a l'aval de barriages—Second meeting of the International Association for Hydraulic Structures Research, Stockholm, 1948, Paper No. 10.

(7) Demarchi G., Professor of Hydraulics at the "Politechico Milane"—Experiments on bed scouring downstream weirs—Second meeting of the International Association for Hydraulic Structures Research, Stockholm, 1948, Paper No. 25.

(8) Mathaway, Gaila., Special Assistant to the Chief of Engineers, Department of the Army, Washington D. C., U.S.A.—Observations on channel changes, degradation, and scour below Dams—Second meeting of the International Association for Hydraulic Structures Research, Stockholm, 1948, Paper No. 27.

(9) Meyer-Peter, E. and Muller, R., Laboratorie de Recherches Hydrauliques anexe a l'Ecole Polytechnique Federale, Zurich.—Affoillements en aval des barrages—Second meeting of the International Association for Hydraulic Structures Research, Stockholm, 1948. Paper No. 29.

(10) Zaki Bey, Hassan, Ph. D., M., Inst. C. E., General Inspector of Irrigation, Upper Egypt and Leliavsky Bey, Serge, Ph. D., M. Inst. C.E.M.A.S.C.E., Directors, Designing Service, Reservoirs and Nile Barrages.—Tail erosion as a factor affecting the safety co-efficient against piping. International Commission on Large Dams, Third Congress, Stockholm 1948, R4.

THE YEAR'S WORK

The following items were discussed at the 1948 Research Committee meeting :—

(1) To study the possibility of reducing scour below the sluices at Bhimgoda weir.
(2) Prevention of erosion at the upstream nose of the right bank Bell *Bund* of the Anderson Weir on the Damodar.
(3) Model experiments for the prevention of erosion at Kurigram in the river Dharla.
(4) Mid-season scouring operations of the right pocket of Sukkur Barrage.
(5) Apron designs for the south guide bank of the Alexandra Bridge on the Chenab—North Western Railway
(6) Scour below weirs—by K. K. Framji.
(7) Model of the bridge on the river Ravi at Madhopur.

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(1) TO STUDY THE POSSIBILITY OF REDUCING SCOUR BELOW THE SLUICES AT BHIMGODA WEIR ([22])

The cost of repairs downstream of the floor below sluices has been mounting up in the past due to concentration of the river at sluices. The abrasion of the floor downstream is particularly severe due to increased velocities of hyper-critical jet bringing down with it mass of boulder in high floods.

This problem was referred to the Central Hydrodynamic Research Station, Poona in the year 1937 and they had recommended *vide* Technical Paper 58 of 1937, a two feet baffle 10·5 feet downstream of the toe of 1 in 20 sloping glacis. They also recommended a two feet deflector sloping up 1 in 5 at the end of the granite floor. These devices were unfortunately never tried. A geometrically similar model to scale 1/40 representing all the six sluices and two weir bays No. 1 and 2, was constructed so that the gauges could be reproduced accurately in a three dimensional flow both upstream and downstream. It was found possible to pull up the standing wave very advantageously for the low floods between 60,000 to 1,50,000, which were the most damaging ones at site. After this the standing wave always formed at the *pucca* floor.

However, as the proposal of silt excluders had already been accepted by the Chief Engineer, it was considered necessary to study these devices of baffles and deflectors in conjunction with the excluders. This very model was now given a vertical exaggeration of 1 to 1·67, and the presence of these devices immediately showed their undesirable result by helping the excluded shingle to pile up on the floor downstream of bay No. 1. It was also seen that the scour downstream was filled up considerably by the action of the excluders in conjunction with the baffle and deflector.

As the problem of securing equitable distribution of flow above has also first to be decided and it is expected that with the modifications proposed therein the action at the sluices downstream would greatly minimise due to

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([22]) United Provinces Irrigation Research Station, Report on Research Progress during 1947, pages 57-58.

reduction in the discharge per foot at the sluices, such devices of baffles and deflectors cannot be studied with accuracy at this stage.

It was decided that no baffle on deflector need be tried at this stage on the downstream floor of the sluices for pulling up the standing wave. This question may be studied after the lowering of crests and the functioning of excluder has been tried at site.

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(2) PREVENTION OF EROSION AT THE UPSTREAM NOSE OF THE RIGHT BANK BELL *BUND* OF THE ANDERSON WEIR ON THE DAMODAR [23]

The erosion behind the upstream nose of right bank Bell *Bund* of the Anderson Weir was caused by the small channel Bodai which takes off from the right bank of the Damodar about three miles above the Anderson Weir, Figure 2 C.14. Previous to the construction of the weir, this channel Bodai used to meet the Damodar somewhere downstream of the present weir. The channel was closed by the afflux *bund* extending from the right abutment. The channel now has made a big embayment behind the Bell *Bund* and after attacking its upstream nose flows over the weir in another big curve. A large island about 600 feet wide has formed in front of the right Bell *Bund*.

An inspection of the site has shown that the embayment on the right bank upstream of the guide bank is due to excess discharge coming down the Bodai from the Damodar. During recent years the course of the River Damodar has changed considerably particularly beyond the point where the Bodai takes off. This change pushes in more discharge through Bodai and in consequence the embayment upstream of the right guide bank now has increased. The Bodai swings round the nose and makes a big curve into the Damodar bed and then flows over the weir at about 600 feet away from the right bank. The Damodar River on the right bank for this width of 600 feet is now almost inactive, and a big island has formed in consequence. To prevent the trouble, attempts were made in the model to put a straight cut through the island on the left of the Bodai, Figure 2 C.15 and to see if the cut develops when the old channel below the offtake of this cut is also closed. It has been found that when the offtake of this cut is placed at A, Figure 2 C.15, the channel develops. If the offtake of the cut is placed at any other point like B Figure 2 C.15 below A, the cut gets silted up at the offtake. With the cut as shown in Figure. 2C.15, the attack on the upstream nose of the Bell *Bund* ceases and the char in front of the Bell *Bund* also gets washed away gradually. If this cut is given and the old channel closed, the dead loop behind the Bell *Bund* will gradually get silted up and there will be no danger of the Bell *Bund* behind washed away or outflanked.

The scour developed this year behind the upstream nose of the right bank Bell *Bund* has gone to a depth of 25 feet below the toe level of the Bell *Bund* at a distance of about 40 feet from the toe, which has been considerably eroded. The embayment has gone 350 feet behind the nose. The survey data of the Damodar and the Bodai from 7½ miles above to 2 miles below the Anderson Weir after the flood season of 1946 has just been received and the model is being laid according to this latest survey data.

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[23] River Research Institute, West Bengal, Annual Report, 1947, pages 67-68.

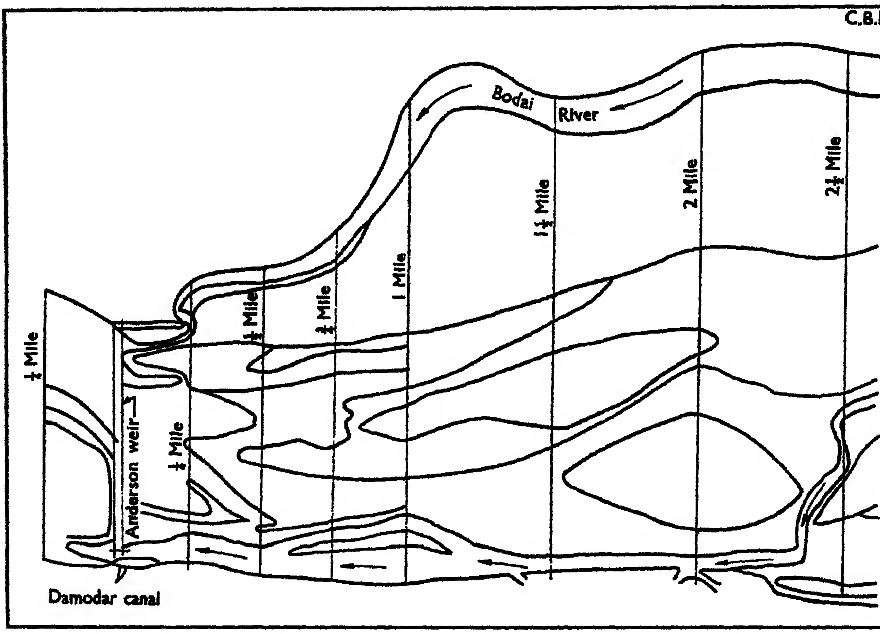

Figure 2C.14:- *Showing plan of Damodar River at Anderson weir (Survey after 1944 flood season)*

Scale 2,000 1,000 0 2,000 4,000 6,000 Feet

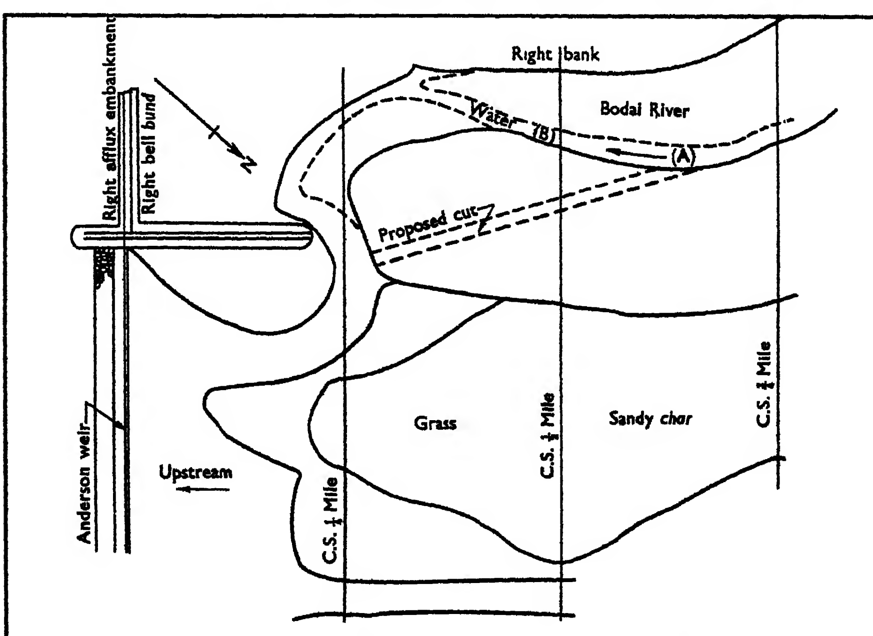

Figure 2C.15:- *Showing plan of the right bank of Damodar near Anderson weir (Survey after 1946 flood season)*

Scale 500 250 0 500 1,000 1,500 2,000 Feet

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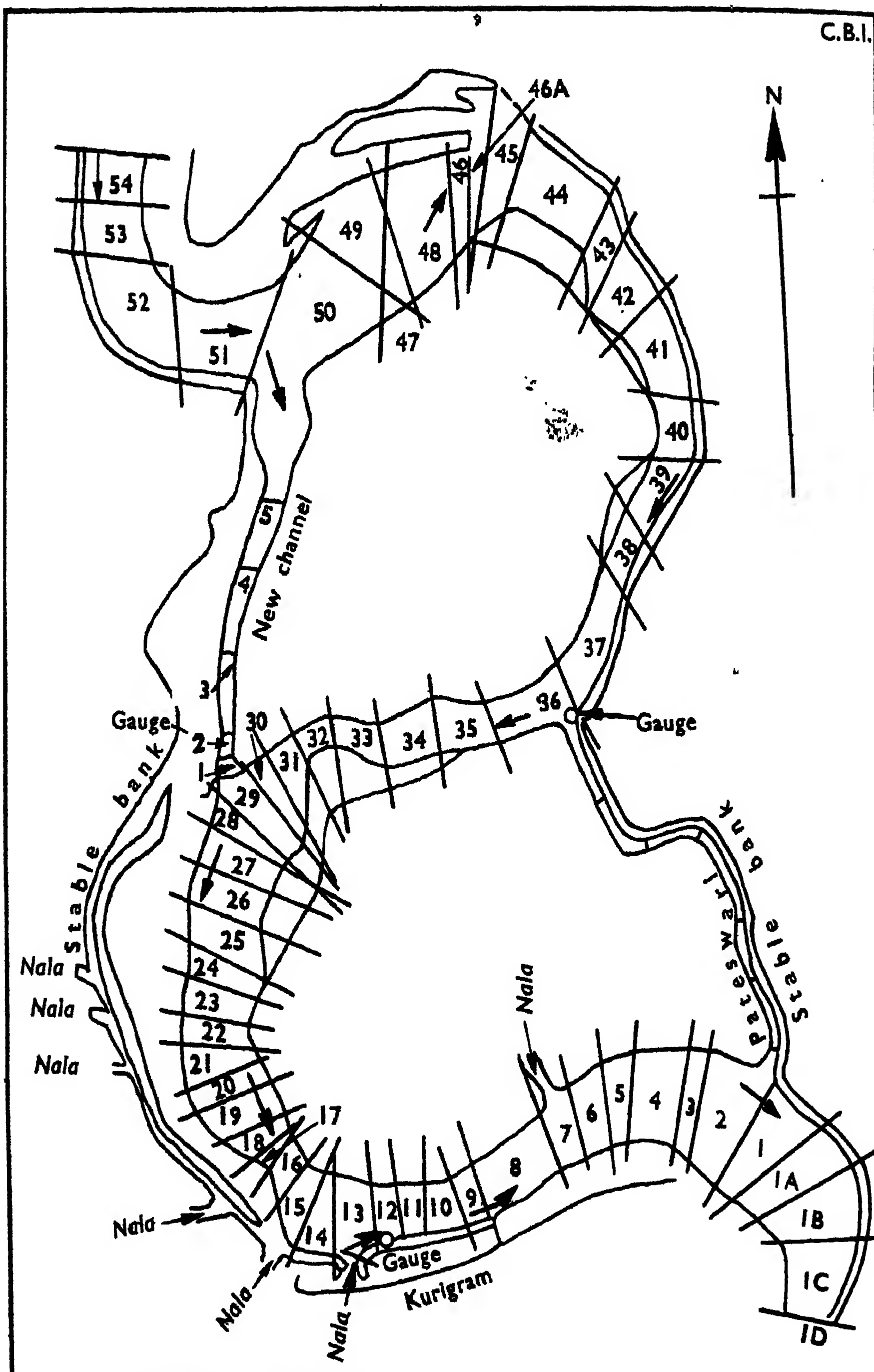


Figure 2C.16:- Showing plan of river Dharla above Kurigram surveyed during July and August 1946

Reg. No. 2737 XDD (C)' 50 1,210 (P.Z.O.).

(3) MODEL EXPERIMENTS FOR THE PREVENTION OF EROSION AT KURIGRAM BY THE RIVER DHARLA[24].

ABSTRACT

These experiments were started for devising measures to prevent erosion of the town of Kurigram by the river Dharla.

The Dharla has a perennial source of supply like the Ganga or the Brahmaputra and has a maximum discharge of about 100,000 cusecs. Its banks at Kurigram are of alluvial soil. Erosion of its right bank at Kurigram started about 10 years ago and became so severe in 1940 that the proposal for shifting the Sub-divisional town from Kurigram to Lalmonirhat (in the district of Rangpur in Bengal) was seriously taken up by Government. In order to protect the town, the Irrigation Department has for the last few years been spending considerable amount of money annually for the construction of a number of small spurs at points of heavy attack on the bank. These spurs are each about 100 feet long and 6 feet wide making about 45° with the bank line downstream, put up by driving salballa posts along the sloping bank to a depth of about 12 feet below the bed and filling the space within the salballa piles by brushwood. The longitudinal profile of each spur slopes down almost parallel to the bank profile, and the nose of the spur on the river bed, though above the water line in the winter season, gets submerged during the flood season and the effectiveness of the whole length of the spur is reduced. The spurs were washed away in most cases during the first flood of the season and erosion continued during the rest of the flood season every year.

To find out if there was any change in the river regime upstream of Kurigram to account for the change of the river at Kurigram, the Director of the Institute with the Superintending Engineer of the Circle went up from Kurigram along the river in a boat and found a new channel falling into the river about 2¼ miles above Kurigram. On local enquiry it transpired that this was a short cut of the meander of the same river opening sometime in 1936 or 1937. It was gradually increasing in size and during the flood season of 1946, its width increased from 200 to 400 feet on the average. Observation during 1946 winter season showed that practically the whole discharge of the river was then coming through this new channel. As the development had a very important bearing on the whole problem, a thorough survey of this river from above the offtake of this new channel to about 1¼ miles below Kurigram passing the point where the dead old course of the Dharla joined the present course Figure 2 C.16 was made during July and August, 1946 and the model was laid according to this survey. Gauge and discharge observations were also taken during 1946 flood season and these were made available to the Institute for running and proving the model. Observations were made at the following three sites :

(1) At Kurigram right bank between Cross-section Nos. 11 and 12.

(2) Within the new channel near its outfall into the Dharla.

([24]) River Research Institute, West Bengal, Annual Report, 1947, pages 68-70.

(3) In the Dharla just above the offtake of the Pateswari, the old Dharla course.

A description of the whole work done on this model is given below :—

(1) Construction of the model was started from the beginning of November 1946 and was completed by the 15th December 1946. The whole portion of the river shown in Figure 2 C.16 was incorporated in the model. The new channel and the Pateswari were also laid as surveyed during July and August 1946. Horizontal and vertical scale ratios were chosen as 1/200 and 1/50, respectively. As the survey was done during the flood season, the banks could not be surveyed, as desired, up to high flood level at all places. Whatever was done was incorporated in the model.

(2) The similarity of the behaviour of the model with that of the prototype was first proved by passing observed discharges over the model and noting the gauge readings, velocities and streamlines obtained. Observations in the model compared well with those of the prototype. The results are given in Table 2 C.7.

(3) The effect of the opening of the new channel was then investigated in the model. The model was run by closing the new channel altogether and by keeping it open as in July and August 1946. Photographs of streamlines were taken in the two cases for two discharges namely :—

(1) Discharge equivalent to 48,000 cusecs in the prototype.

(2) Discharge equivalent to 70,000 cusecs in the prototype.

As the new channel developed the point of attack gradually shifted towards the upstream and Kurigram proper was affected. With the new channel as in July and August 1946, the attack was on a point just above Kurigram town proper. This was also evident from the prototype-survey of the right bank after 1946 flood season—which had shown that there was considerable silting at the right bank on the town proper.

(4) It was found in the model that the 14 or 15 spurs which were being put up in previous years for protecting the bank from erosion were defective in design. The crest of these spurs sloped down parallel to the bank and nearly the whole length of the spur below the bank would be submerged with rise in water level. The spur would have been more effective, if the top of the whole length of the spur be kept above the highest water level. It was found in the model that only three spurs constructed in this way would be able to stop erosion on the banks at Kurigram. These spurs are to be made of earth dumped on the river bed and protected on the sides by stone pitching and provided with stone falling apron. The following cases were tried in the model :—

(1) A spur near cross-section No 13 and a spur near cross section No. 11.
(2) A spur near cross section No. 12 and a spur near cross-section No. 11.
(3) A spur near cross-section No. 13, a spur near cross-section No. 12 and a spur near cross-section No. 11.

Case (1) could not perevent direct attack on the bank between the two spurs. Case (2) could not prevent attack on bank above the spurs. Case (3) was successful in eliminating direct attack on the bank at all places.

In all cases the current was thrown away from the bank below C.S. No. 11. Thus only 3 spurs if properly constructed will be able to protect the town of

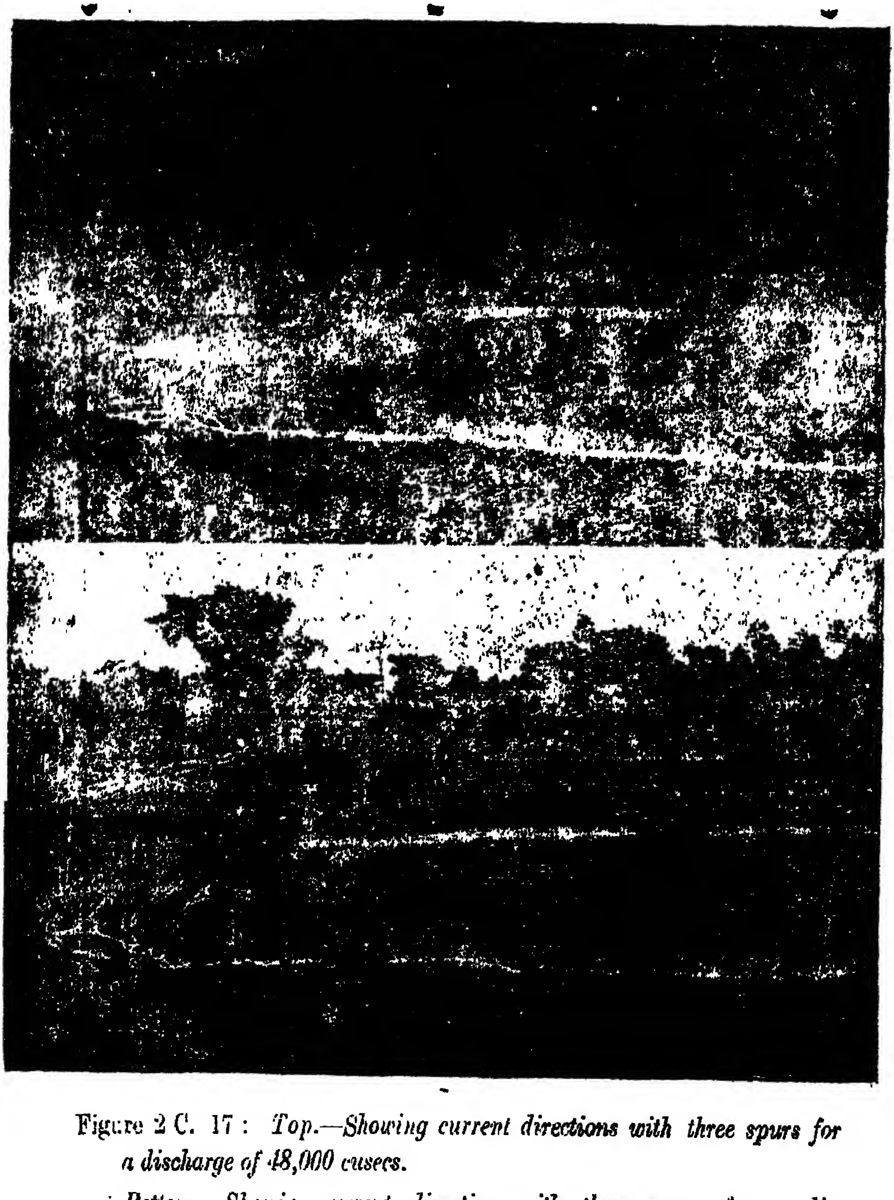


Figure 2 C. 17 : *Top.—Showing current directions with three spurs for a discharge of 48,000 cusecs.*

Bottom.—Showing current direction with three spurs for a discharge of 70,000 cusecs. Dharlo River model.

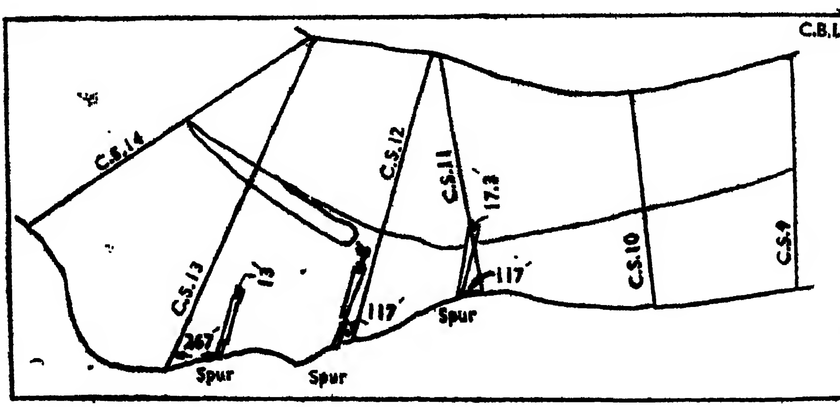
Kurigram from further erosion. The dimensions of spurs of case (3) above are given below :—

(1) Spur near cross-section No. 11 projected 100 feet into the river bed from the bank line (1946) making an angle of 17° 30′ with cross-section No. 12 towards the downstream.

(2) Spur near cross-section No. 12 projected 100 feet into the river bed from the bank line (1946) making an angle of 8° with cross-section No. 12 towards the downstream.

(3) Spur near cross-section No. 13 projected 50 feet. into the river bed from the bankline (1946) making an angle of 13° with cross-section No. 12 towards the downstream.

Position and orientation of these spurs are shown in Figure 2 C-18. The nature of the scour obtained in the model along the nose of the three spurs fter running a maximum discharge of 70,000 cusecs is shown in Figure 2 C.19



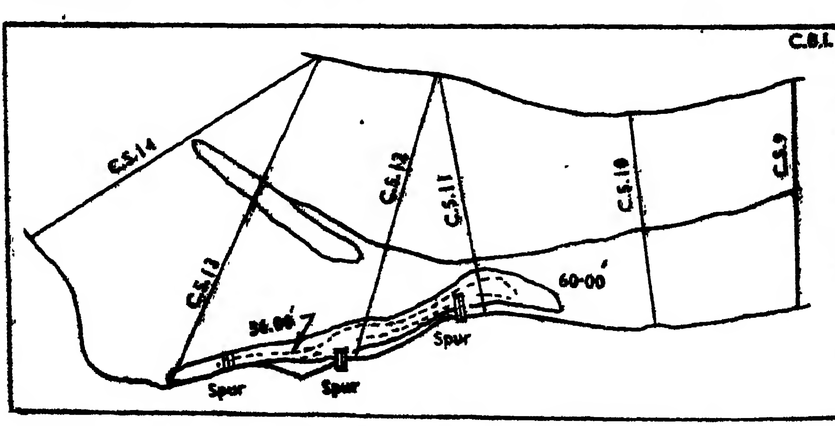


Figure 2 C.18 : *Top—Showing exact position of the spurs tried in the model.*

Figure 2 C.19 : *Bottom—Showing control lines of the scour produced at the nose of the spurs in the model.*

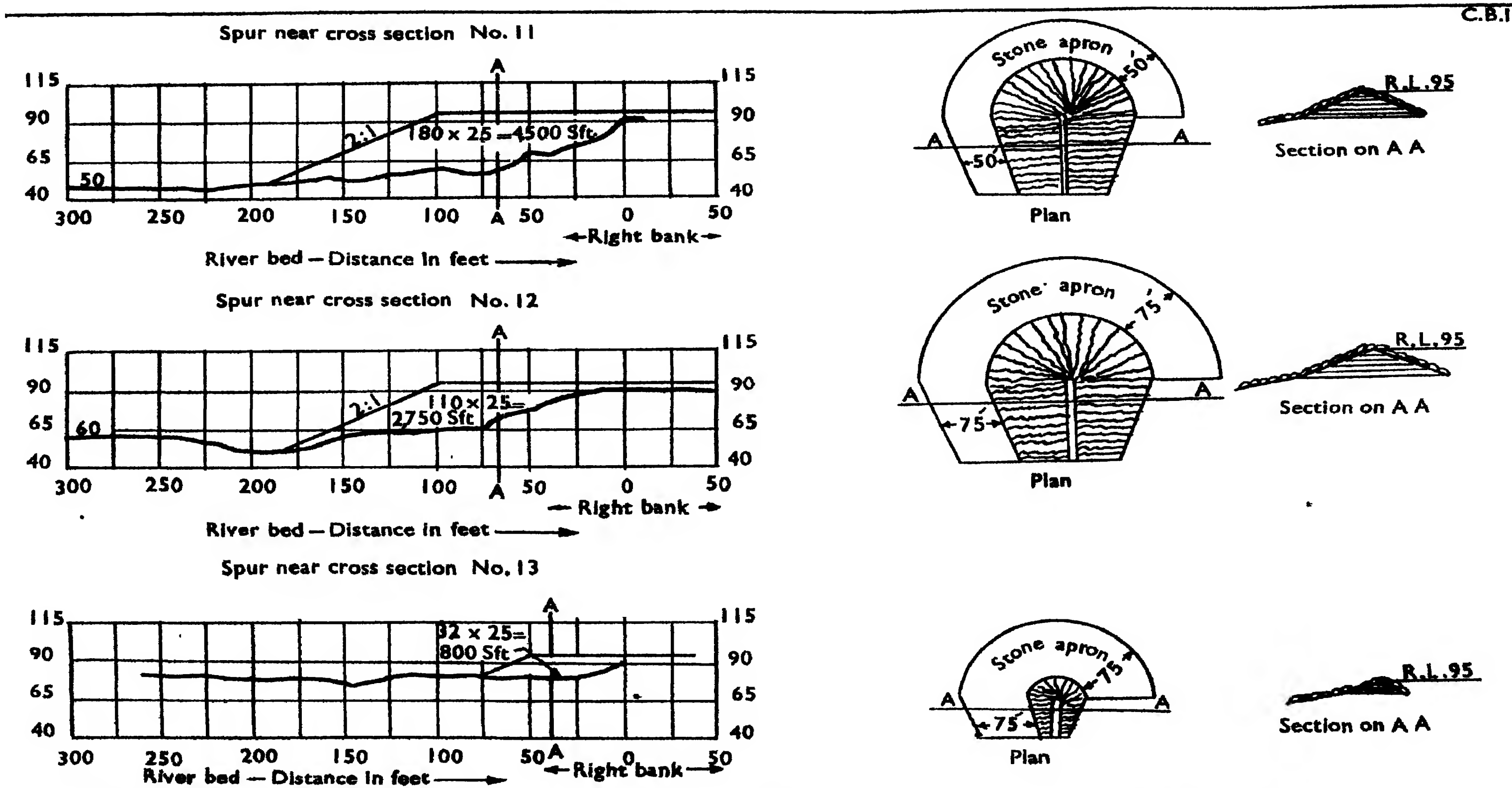


Figure 2C.20:-*Sections of the proposed Spurs on the right bank of the Dharla River at Kurigram*

No. 2737 XDD (C)' 50-1,210 (P.Z.O.).

The maximum water level observed at Kurigram during the years 1944, 1945 and 1946 was 88·80 (R.L.). The top level of the spurs should be five feet above the observed maximum water level of Kurigram. The top level has, therefore, been given to be 95·00 (R.L.), and width of the spur at the top is 10 feet. The sides pitched with stone has been given a slope of 2 : 1. Falling apron has been provided to cover the maximum depth of scour observed in the model. The details of the spurs have been shown in Figure 2C. 20. The amount of earthwork in the three spurs will be about 400,000 ; 200,000 and 32,000 cubic feet respectively.

TABLE 2C.7

Dharla River Model

Comparisons of results of Model with prototype values gauge reading (R. L.).

Discharge cusecs	At Kurigram		In the new channel		At Pateswari Offtake	
	Model (Feet)	Prototype (Feet)	Model (Feet)	Prototype (Feet)	Model (Feet)	Prototype (Feet)
23,000	80·35	80·35	81·25	81·27	82·35	81·58
29,000	83· 2	83·25	84· 0	84·17	84· 7	84·54
32,000	83· 5	53·45	84· 4	84·38	85· 5	84·68

(4) MID-SEASON SCOURING OPERATIONS OF THE RIGHT POCKET OF SUKKUR BARRAGE[25]

ABSTRACT

The Sukkur Barrage authorities have had under consideration the proposal of having additional scouring operations of the Right Pocket of Sukkur Barrage in the months of June, July and August without closing the Canals, in the hope that these flushings will keep the pocket clear and thus provide a silt trap where silt may deposit instead of being carried into the canals. The note discusses whether such scouring operations are necessary or likely to be useful.

(25) Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report, Technical, 1947 pages 61-64.

(5) APRON DESIGNS FOR THE SOUTH GUIDE BANK OF THE ALEXANDRA BRIDGE ON THE CHENAB—NORTH WESTERN RAILWAY(26)

The cold weather survey of 1947 of the Chenab river at Alexandra Bridge revealed that the apron of the left guide bank between chainages 6 to 9 and at chainage 12 had launched to a slope steeper than 2 : 1 for which it was originally designed.

With parallel flow conditions, the stone pitching, generally, is expected to assume a side slope of 2 : 1. In the present case, however, the main current after leaving the Pindi Spur nose heads directly to the south bank, thus creating severe curved flow conditions. The profile of the guide bank generally associated with such conditions in itself, should not be a source of anxiety unless accompanied by other signs of weakness or instability, such as sufficiency of stone thickness of the launched apron.

With curved flow, conditions similar to those at the head of a large-radius guide bank are obtained and as maximum scour in such a case is observed to be of the order of $2R$ (Lacey), the apron should be designed for scour below high flood level $= 2R$, (R being the Lacey normal depth)—and which in this case $=$ 45 feet below low water level.

It was, thererfore, recommended that—

(1) provided there are no other signs of weakness, it is not essential to endeavour to maintain the launched apron to the unnatural slope of 2 : 1 in continuation of the permanent slope ; but

(2) additional stone apron should be provided up to a distance of (*a*) 68 feet ($1 \cdot 5 \times 45$) from the present toe of the launched arpon or (*b*) to the bottom of the scour if it be deeper than 26 feet 8 inches below low water level—$1 \cdot 5\ R$ (Lacey)—whichever is more.

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(6) SCOUR BELOW WEIRS (27)

by

K. K. Framji, M. B. E., I. S. E.

Director, Central Waterways, Irrigation and Navigation Research Station, Poona.

Not discussed as per decision of the Research Committee.

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(26) Central Waterways, Irrigation and Navigation Research Station Poona, Annual Report Technical, 1947, page 65.

(27) Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report Technical, 1947, pages 67-82.

(7) MODEL OF THE BRIDGE ON THE RIVER RAVI AT MADHOPUR[28]

At the instance of the Superintending Engineer, Central Public Works Department, Madhopur, a model of the proposed bridge on the river Ravi downstream of Madhopur Headworks was constructed for studying the conditions of flow in relation to various portions of the headworks *viz.*, undersluices, shutter, weir, river creeks and high right side.

The discharge in this case is mainly concentrated in the undersluices Piers No. 1 and 2 are, therefore, likely to be under heavy action. Preliminary investigaion of the model showed that since no general scour occurred between the undersluice cistern and the bridge piers, the proposed protection of that area was unnecessary. There was only local scour at the pier noses.

DISCUSSION BY THE RESEARCH COMMITTEE

The Chairman (S. Man Singh) wanted to know whether in view of the fact that item (6) having been presented at the meeting of the International Association of Hydraulic Structures Research it should be discussed here or not.

The Committee wished to discuss it, but not bring it on record.

Mr. S. N. Gupta introducing item (1) said that during low floods (of the order of 60,000 to 1,50,000 cusecs) in the Ganga, the standing wave always formed below the floor of the undersluices at Bhimgoda weir as a result of which a deep scour had developed downstream endangering the safety of the whole structure.

Remedial measures in the form of a baffle and a deflector to be provided on downstream floor were recommended by the Poona Station in 1937 but were not tried then. A geometrically similar part model of the undersluices and the weir to scale 1/40, indicated the efficacy of these devices.

In view of the shingle trouble in the new supply channel, a shingle excluder was recommended to be constructed in bay No. 1 of the undersluices which would pass the shingle downstream of the sluices to the extent that no remedial measures need be necessary in addition. Again the equitable distribution of flow over the weir and the sluices would relieve the latter of the excessive

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([28]) East Punjab Irrigation Research Institute, Amritsar, Annual Report 1947, page 20.

strain. The effect of these two proposals would be studied in the prototype and further remedial measures suggested, if need be, at a later stage.

DR. N. K. BOSE said that he had nothing to add to what was given in the report regarding items (2) and (3).

RAO BAHADUR D. V. JOGLEKAR introducing item (4) said that the Sukkur Barrage authorities had under consideration the proposal of additional scouring operations of the Right Pocket of Sukkur Barrage in the months of June, July and August without closing the canals, in the hope that these flushings would keep the Pocket clear and thus continuously provide a silt trap where sand might deposit instead of being carried into the canals.

There was no progressive reduction of area in the Pocket as would threaten the working of the right bank canals. Original bed levels were resorted after the September scouring.

Flushings in June to August were, therefore, not necessary nor would they materially affect the sand charge in the canals, as the additional cross-sectional area obtained in the Pocket after scouring would be quickly filled up. Moreover, the suggestion that the scouring should be done *with canals open* was demonstrably undesirble— as the model clearly showed—because heavy sand charge was thereby drawn into the pocket, a large part of which entered the canals.

Introducing item (5) he said that the cold weather survey in 1947 of the Chenab river at Alexandra Bridge revealed that the apron of the left guide bank between chainage 6 to 9 and at chainage 12 had launched to a slope steeper than 2 : 1 for which it originally designed. The Director's opinion regarding the necessity of providing additional stone to ensure adequate protection to the guide bank was, therefore, sought.

With parallel flow conditions, the stone pitching was expected generally, to assume a side slope of 2 : 1. In the present case, however, the main current after leaving the Pindi Spur nose or right bank about one mile upstream of the bridge, flowed across towards the left bank upstream of the left guide bank creating severe curved flow conditions. The profile of the guide bank generally associated with such conditions was curved in cross-section, as in the present case. This, in itself, should not be a source of anxiety unless accompanied by other signs of weakness or unstability such as insufficiency of stone thickness of the launched apron.

With *curved flow*, conditions similar to those at the head of a long-radius guide bank were obtained and as maximum scour in such a case was observed to be of the order of 2 R Lacey, the apron should be designed for scour below highest flood level $= 2R$, R being the Lacey normal depth which in this case $=45$ feet below low water level.

It was, therefore, recommended that :—

Provided there were no signs of weakness, it was not essential to endeavour to maintain the launched apron to the unnatural slope of 2 : 1 in continuation of the permanent slope; but additional stone protection should be provided.

Regarding item (6) MR. D. V. JOGLEKAR had nothing to add.

DR. H. L. UPPAL introducing item (7) said that there was really no necessity of carrying on model experiments. Due to the high floods last year on the Ravi which had a discharge of 567,000 more than the maximum recorded before, they had to carry out certain experiments, which had been presented.

MR. GOVINDA RAO referring to item (6) observed that the note was not of as general and wide an application as the heading suggested. The remarks seemed to apply particularly to low weirs and barrages in deltaic regions.

The nature of flow below a bridge, a weir and a barrage had to be distinguished before examining the possibility of applying or evolving a formula for indicating maximum scour. In a bridge, the flow was through all the vents all the time. The intensities of flow, that is, the dominant discharge per foot width of waterway was dependent only on the intensity of flood. In the case of a river whose regime had been fully established, the scour depended mainly on the intensity of discharge. The flow pattern was constant. It was the same case in an open weir with no gates on it. Due to heading up of water upstream and downstream water levels differed perceptibly unlike a bridge. The scour depended considerably on tail water conditions of flow. In a barrage, the flow was controlled through gates so that there could be variation in discharge intensity per foot width of waterway, in general pattern of flow and also in tail water conditions. The depth of maximum scour had, therefore, to vary in all the three cases due to very fundamental differences mentioned above.

Regarding the causes of scour, in addition to the three causes *i.e.*, excess energy of hyper-critical flow not being dissipated, formation of standing wave, beyond the hard apron floor and creation of turbulence due to high bed velocities created by high pavements of bridge floors, it might also be due to turbulence caused by concentration of flow in particular portions as it happened if a weir was built immediately below a curve in a river, formation of eddies caused by return flow at masonry guide walls, training walls *etc.*, energy of shooting jets of waters as when a siphon in a weir was discharging, generation of vortices at points of sudden changes in direction and uplift created due to high

velocities at exit gradients of pavements *etc.*

In the Lacey formula $D = 0 \cdot 9 \ (\frac{q^2}{f})^{\frac{1}{3}}$, D is the depth below maximum water level, q is the discharge intensity and f is the silt factor generally taken as equal to $1 \cdot 76\sqrt{m}$ where m is the bed diameter. In this formula q is merely the discharge intensity. It took no account of the nature and strength of turbulence causing the scour *i.e.*, whether it was a vortex, positive or negative a shooting jet or a turbulence caused by insufficient dissipation of energy at a standing wave. It did not also take into account the nature and amount of silt charge in the discharge. As mentioned by the author it did not also take into account the effect of enlargements accruing in narrow channels.

The factor " f " in the formula defined the nature of bed material. If the bed material was very small in size say less than $0 \cdot 5$ mm., it was generally admitted that the size of the material affected the maximum depth of scour. In bed materials of bigger sizes Butcher and Atkinson (Proceedings of Am., Soc. C.E., Vol. 235 page 256) had shown in this paper published by them, that the depth of scour was independent of the nature of bed material above $0 \cdot 5$ mm. size. So D was not always a function of $(\frac{1}{f})^{\frac{1}{3}}$.

The best method for determining the maximum depth of scour was, therefore, by conducting model experiments. But in interpreting their results, the limitations these experiments had with those in prototype should not be forgotten. In addition to the limitations so clearly mentioned by the author, there were the limitations set up due to (1) difference in cohesive force between particles forming the bed material in the prototype and in the model and (2) that due to strength of a vortex not being scalar.

THE SECRETARY referred to item (6) and said that in his opinion, it was wrong to republish this paper in the Annual Report of any Institution. In any case, he did not propose to publish it in the Annual Report of the Board wherein only original articles or items of research carried out were published and he thought that the Research Committee should be very clear about it. It was a practice in Poona, however, to publish such papers in the Annual Reports.

After discussion, it was agreed that such papers as had been published elsewhere should not be reprinted in the Annual Report of any station or of the Board. There should be no objection, however, to give a reference to it.

Mr. C. V. Gole stated that the scour downstream of the sluices of the Bhimgoda Weir occurred mainly due to the hyper-critical flow persisting below the sluices, as the glacis slope was very flat *viz.*, 1 in 20. It was necessary to have the standing wave at the toe of the glacis which could be done by steepening the glacis slope to, say, 1 in 5 and a deeper cistern.

Mr. S. N. Gupta answered that he had already mentioned that they would take up all proposals of putting up a suitable glacis and other devices below the sluices at a later stage when they had studied the result of their proposal already arrived at by model experiments, in the prototype.

The President wanted Rai Bahadur Joglekar to explain the position regarding the collection of data on scour below weirs.

Rao Bahadur D. V. Joglekar said that the collection of data was still proceeding. Whatever had been collected had been published in the Report.

DISCUSSION BY THE BOARD

The Secretary said that seven items were discussed at the Research Committee meeting (page 663). Item (6) was also contributed to the International Association of Hydraulic Structures Research This item was not discussed and the decision of the Research Committee in this connection was on page 674.

Rai Bahadur Brij Narayan enquired whether any further progress had been made regarding " Scour and erosion in Rivers ". He was specially interested on this subject as it affected bridge piers. He stressed the necessity of collecting data during and after floods.

The Secretary said that the data so far collected had appeared in the Annual Report of the Central Waterways, Irrigation and Navigation Research Station.

Rai Bahadur C. L. Handa explained what happened at the Naushera site of the bridge over Bias last summer. The river had been flowing on the right side for long long time. The design of the bridge was based on the latest surveys and there was no apprehension of any serious change in the course of the river. But the river changed its course last year. Similarly in the case of the Harike site which Rai Bahadur Brij Narain mentioned they had two maps of the Survey of India—one dated 1912 and the other dated 1937 and

when they tried to compare them together they found that the confluence of the river showed a discrepancy of half a mile. If this rate of the moving down of the confluence of the river continued at the same speed he observed that the site might be obsolete in 20 years.

THE CHAIRMAN (MR. S. A. GADKARY) opined that the construction of barrage would solve it.

The Board accepted the decision of the Research Committee.

(iv) Spill

PRELIMINARY NOTE

This sub-head was introduced for the first time in 1945. To clarify the objective of investigation under the subject the Board at its 1945 annual meeting adopted the following resolution :—

" Resolved that the following questions in conclusions with spill need investigation :

(1) The merits of contorlled spill with regard to the effect on the flooded areas and the regime of the river downstream.

(2) The discharge and duration of flow required to flush drainage channels under controlled spill.

(3) The effect of spill on the sand transporting capacity of parent channels.

And that the following data should be collected :—

(*i*) Gauge readings and discharges with cross-sections, water surface slopes, angles of off-take, and other hydraulic data in parent rivers.

(*ii*) Silt loads of parent and spill channels.

(*iii*) Rate of accretion in river and flooded area."

The following item was discussed at the 1947 Research Committee meeting :—

(1) Model experiments on the revival of river Chandana.

THE YEAR'S WORK

The following items were discussed at the 1948 Research Committee meeting :—

(1) Model experiments for the revival of the river Chandana.

(2) Model experiments in connection with the revival of the Peali River.

(3) Matla—Peali Tidal Model—Analysis for Data.

(1) MODEL EXPERIMENTS FOR THE REVIVAL OF THE RIVER CHANDANA[29]

ABSTRACT

The experiments on this model were in the beginning carried out at the Model Station at Belghoria and the report of the whole work was published in the Annual Report of 1946, where history and other details of the problem were also explained. Describes further experiments carried out.

[29] River Research Institute, West Bengal, Annual Report, 1947, pages 70-74.

The Chandana, in the district of Faridpur, in East Bengal, is a small spill channel of the Padma barrely 150 feet wide and takes off from the right bank of the Padma at a distance of about 38 miles downstream of the Hardinge Bridge (Bengal Assam Railway). It is below the offtake of the Gorai, another spill channel of the Padma carrying a maximum discharge of about 200,000 cusecs. The Chandana, however, carries a maximum discharge of not more than 2,000 cusecs only. Previous records show that about the year 1863 the main course of the Padma was along its right bank near the offtake of the Chandana. At present, the main course of the Padma near this offtake is along the left bank. There is a big island just in front of the offtake, see figure 2 C. 21 and during low stages of the Padma it is only the back-water coming

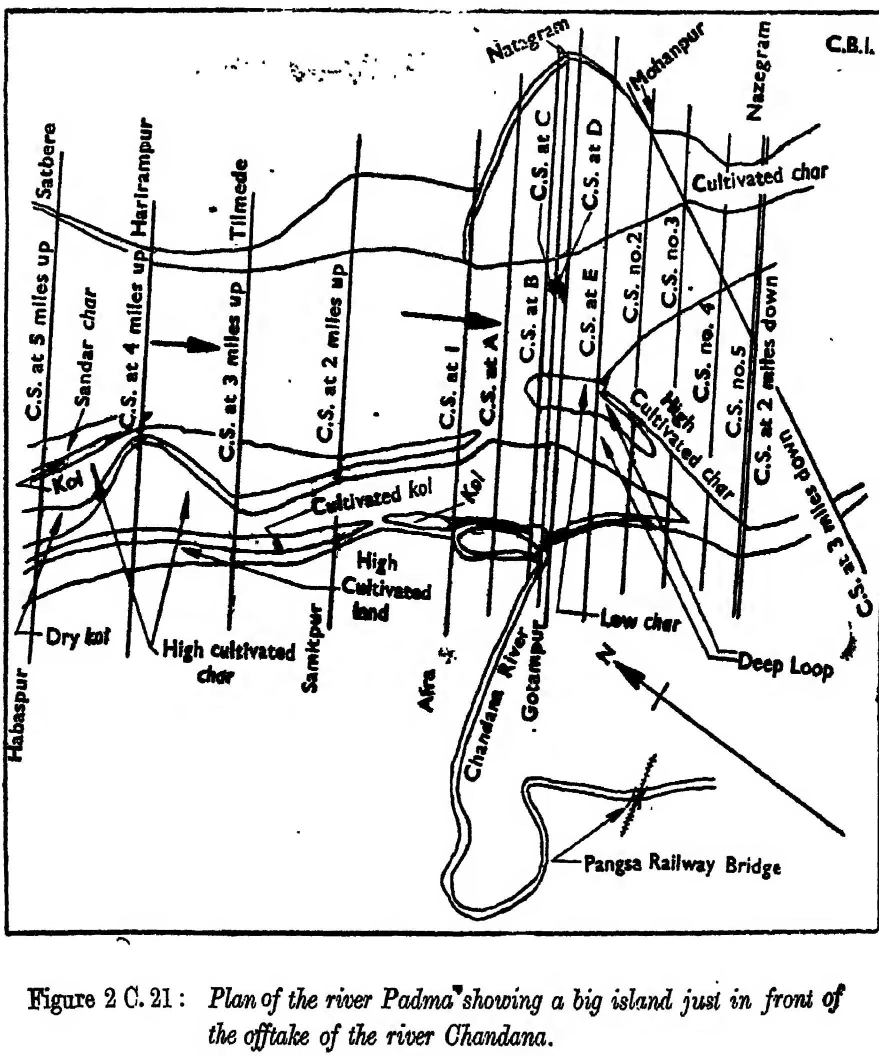

Figure 2 C. 21 : *Plan of the river Padma showing a big island just in front of the offtake of the river Chandana.*

through a small loop from the downstream end of this island along the right bank that enters the Chandana. As discharge in the Padma increases, this island gets submerged and direct entry into the Chandana is obtained. The Chandana, however, like many other spill channels of the Padma, has badly silted up and the surrounding area which was once healthy and prosperous has also decayed. This spill channel now gets supply from the Padma only during the flood season and for the remaining part of the year it becomes a series of stagnant pools. The problem now is to revive this dying spill channel. A project for excavating the Chandana to designed sections has been proposed but as it is not sure if the resectioned Chandana will not silt up again, the problem was sent to the Institute for investigation.

AVAILABLE DATA

The only data available for construction of a model was the 1946 (February to April) survey of the Padma from five miles above the offtake to three miles below it and the 1943-44 survey of the Chandana.

The data available for running and proving the model were—

(1) Discharge and gauge observations at the Hardinge Bridge at Paksey about 38 miles above the offtake of the Chandana at Gautampur.

(2) Gauge readings on the left bank of the Padma at Dogachia about 16·5 miles above the offtake at Gautampur.

(3) Gauge readings of the Padma at the offtake of the Chandana at Gautampur.

(4) Gauge readings of the Padma at Goalundo, about 23·5 miles below the offtake at Gautampur.

(5) Gauge and discharge observations of the Chandana at the Railway bridge over the Chandana at Pangsa 38,000 feet from the offtake at Gautampur.

Simultaneous data for all the above five items were available only for the year 1945.

From (1), (2) and (3) above the gauge readings of the Padma at a distance of five miles above Gautampur and from (3) and (4) above gauge readings of the Padma at a distance of five miles below Gautampur were interpolated. The discharge of the Padma at five miles above Gautampur was obtained by deducting the approximately known Gorai discharge from the Padma discharge at Hardinge Bridge, there being no tributary or spill of the Padma in this reach.

THE MODEL

The model built at the Model Station of the Institute at Belghoria and the experiments conducted therein were given in detail in the Annual Report of 1946. The difficulty there was that discharge more than 8·5 cusecs could not be obtained with the help of the pumps installed there. The scale of the model there had to be reduced to $\frac{1}{1000}$ horizontal and $\frac{1}{80}$ vertical and the maximum discharge in the Padma that could be run in this model was only 850,000 cusecs. The chances of increasing the pumping capacity at Belghoria being not immediately encouraging it was decided to build a similar model on a bigger scale at the Galsi Model Station of the Institute where higher discharges were available direct from the Damodar Canal.

The portion between five miles above and three miles below the Gautampur offtake of the Padma was laid in sand (average diameter=0·33 mm.) on a scale $\frac{1}{500}$ horizontal and $\frac{1}{40}$ vertical at the Galsi Model Station. On the upstream end of the model another 5 miles were contoured according to the aerial survey of 1939. This was done to simulate proper approach conditions into the model. Similarly at the downstream end a reach of two miles more was contoured on earth. The bed and banks of the Chandana up to 40,000 feet from the offtake (that is, up to 2,000 feet below the Pangsa Railway Bridge where prototype gauge and discharge observations were available) were made in weak cement plaster placed over properly sectioned earth, because the problem of study was the entry of discharge and silt from the Padma into the Chandana and not the bed movement and erosion of the Chandana itself. The bed of Chandana does not contain any sand, it is hard non-erodible. The right bank of the Padma above the Chandana was properly contoured according to 1938-40 survey data to allow proper spilling in the model over the right bank. This portion of the right bank is low and becomes submerged when the discharge in the Padma goes above 800,000 cusecs. Part of this discharge enters the Chandana over its right bank and increases its discharge.

Gauges in the Padma were put in the model (1) at five miles above Gautampur, (2) at Gautampur and (3) at five miles Below Gautampur. In the Chandana the gauge was measured at the Pangsa Railway Bridge. The discharge in the Chandana was measured in the model below the Pangsa Railway Bridge by the volumetric method.

THE EXPERIMENTS

The construction work of the model was completed towards the end of July 1947. The data available for building this model were rather incomplete and various adjustments had to be done in the model, before it could be made

to reproduce prototype conditions. The work took the whole of September 1947. Table 2 C. 8 was prepared from observed gauge readings and discharges of the Chandana and the Padma during the year 1945 and the model was expected to reproduoe these readings.

Table 2 C. 9 shows the readings observed in the model compared to those in the prototype. Except the discharge figures of the Chandana, which were not very accurate for the prototype, other figures tallied well. The model was, therefore, taken to have reproduced prototype conditions.

Experiments were conducted to find the amount of silt entering the Chandana from the Padma for different discharges of the latter in a fixed period of time in the model. In each experiment a steady discharge was passed over the model and gauges were reproduced according to the prototype values by adjusting the downstream gates of the Padma and the Chandana. The model was run in this steady stage for seven hours in each experiment and the volume of sand depositing on the Chandana bed was collected and mesasured. No sand was found to have reached the Pangsa Railway Bridge or the downstream measuring chamber of the Chandana. Table 2 C. 10 gives the results of these experiments. Results of these experiments were not found to be consistent and it took nearly two months to find the cause of this inconsistency. In the early part of these experiments—in September and October 1947—the water of the canal, passed over the model, was turbid with fine suspended clay. Nothing could be seen below the surface and this fine suspended clay would cover nearly the whole bed after each experiment. This scum had to be removed after each experiment and the bed recontoured. In November 1947 when the canal water became clear it was found that there was practically no bed movement in the model near the offtake of the Chandana. The surface velocity in the Padma near the offtake was found not to exceed 0·3 feet per sec. in the model even at the highest stage. The sand that was found to have entered the Chandana in the above experiments had all come there as floating particles when water was first passed over the dry recontoure model bed. As soon as these floating particles of sand would pass the first constriction in the Chandana, they would get mixed up with water and settle down immediately after the constrictions. This was confirmed in the first week of December 1947 when similar experiments were repeated without allowing the surface floating sand to enter the Chandana. The results of experiments on December 4 and 5, 1947 will prove this point. The natural conclusion was, therefore, that no sand was found in the model to have entered the Chandana from the Padma, with the present conditions of the Chandana.

TABLE 2 C. 8

Simultaneous gauge readings and discharges observed in the Padma and Chandana during 1945.

Date (1945)	Time	Gauges of Padma			Gauges of Chandana	Discharge		
		5 miles upstream	Gautampur	5 miles downstream	Pangsa	At Hardinge Bridge	At Gautampur	At Pangsa
July 5 ..	..	..	..	..	..	279,000	239,200	..
July 9 ..	..	31·54	30·25	28·87	28·00	543,000	465,500	..
July 14 ..	..	31·34	30·05	28·67	27·90	495,000	424,300	..
July 19 ..	..	33·72	32·30	30·89	29·70	773,000	663,000	..
July 25 ..	..	34·37	32·95	31·73	30·40	863,000	740,000	..
July 31 ..	..	34·04	32·75	31·51	30·30	857,000	735,000	1,025
August 4 ..	..	34·45	33·05	32·00	30·45	901,000	773,000	..
August 9 ..	..	35·09	33·65	32·43	31·10	964,000	827,000	..
August 15 ..	..	34·91	33·65	32·49	31·20	847,000	726,000	837
August 20 ..	..	34·96	33·70	32·53	31·40	850,000	727,000	..
August 25 ..	..	35·93	34·60	33·32	32·00	981,000	841,000	..
August 30 ..	..	36·65	35·15	33·71	32·75	1,205,000	1,032,000	1,125
September 5	..	36·91	35·20	33·71	33·20	1,227,000	1,043,690	..
September 10	..	36·59	34·95	33·66	33·40	1,182,000	1,013,000	..
September 15	..	35·85	35·15	33·70	33·40	1,260,000	1,080,000	1,558
September 19	..	37·20	35·50	34·03	33·90	1,410,000	1,209,000	..
September 25	..	6·67	35·05	33·57	33·80	1,369,000	1,173,000	..
September 29	..	34·97	33·60	32·17	32·50	1,115,000	956,000	..

TABLE 2 C. 9

Chandana Offtake Model

Prototype and Model observations (converted to prototype values) compared.

Date of Experiment (1947)	Observation in	Discharge in cusecs	Water level (R.L.) of the Padma at			Water level of the Chandana at Pangsa Rly. Bridge	Discharge of the Chandana at Pangsa Rly. Bridge in cusecs
			Five miles upstream	Gautampur offtake	Five miles downstream		
September 19	Model ..	465,500	31·4	30·0	28·8	28·0	850
	Prototype ..	465,500	31·54	30·25	28·87	28·0	Not observed.
September 23	Model ..	735,000	34·0	32·6	31·6	30·32	1,050
	Prototype ..	735,000	34·0	32·75	31·51	30·30	1,025
October 1 ..	Model ..	1,080,000	36·8	35·4	33·8	33·4	1,750
	Prototype ..	1,080,000	36·6	35·15	33·7	33·4	1,558
October 21 ..	Model ..	827,000	35·2	33·8	32·4	31·1	1,700
	Prototype ..	827,000	35·09	33·65	32·43	31·10	Not observed.
November 2	Model ..	1,209,000	37·2	35·7	32·6	33·8	1,600
	Prototype ..	1,209,000	37·20	35·50	34·03	33·9	Not observed.

TABLE 2 C. 10

Chandana Offtake Model Experiments.

Date of Experiment (1947)	Discharge in the Padma below the Garai offtake in cusecs		Discharge in the Chandana below the Pangsa Rly. Bridge in cusecs	Volume of sand entering in 7 hours of the experiment in c. c.	Remarks
	Model	Prototype	Model	Model	
September 18 ..	9·20	465,500	0·017	Nil	
September 23 ..	14·52	735,000	0·021	100	
September 30 ..	14·52	735,000	0·020	Nil	
October 1	21·35	1,080,000	0·035	90	
October 4	14·52	735,000	0·020	130	
October 11	14·52	735,000	0·030	125	
October 14	21·35	1,080,000	Not measured	75	
October 20	30·00	1,517,000	0·050	35	
November 6	23·89	1,209,000	0·032	35	
November 7	23·89	1,209,000	0·037	55	
November 21 ..	16·34	827,000	0·034	60	
November 25 ..	14·52	735,000	0·027	50	
December 4	14·52	735,000	0·024	Nil	In these experiments floating sand was not allowed to enter the Chandana.
December 5	14·52	735,000	0·025	Nil	

It was also found that the velocity of entry of water into the Chandana at its offtake increased gradually up to the bankfull stage of the Padma (corresponding to a discharge of about 900,000 cusecs at the Hardinge Bridge) but as spilling started over the right bank of the Padma above the Chandana offtake, part of this spill-water entered the Chandana over its right bank and backed up the flow in the head-reach of the Chandana ; consequently the slope of the Chandana within the first five miles from the offtake became flatter and flatter, as the discharge in the Padma increased over its bankfull stage. To detect this flattening of slope, a gauge was put in the Chandana about four miles below the offtake and the gauge readings were observed for different discharges at different gauge sites. The observed readings are given in Table 2C. 11

TABLE 2 C. 11

Discharge in the Padma below the Gorai offtake		Gauge readings (R.L.) observed in the model at		
Model (cusecs)	Prototype (cusecs)	Gautampur	Four miles below Gautampur	Pangsa Railway Bridge
9·20	465,500	30·25	29·0	28·0
14·50	735,000	32·8	31·8	30·3
21·35	1,080,000	35·2	34·6	33·4
30·00	1,517,000	36·0	35·8	34·5

Experiments so far carried out have gone to prove that at the present position of the offtake there is no bed sand entry into the Chandana from the Padma at any stage of its flow. This is also corroborated by the bed survey of the Chandana from its offtake undertaken in 1945 by the River Research Institute, Bengal. This survey showed that the bed material was every where clay and very fine silt. No sand was found anywhere in the whole reach of the river from Gautampur downwards.

(2) MODEL EXPERIMENTS IN CONNECTION WITH THE REVIVAL OF THE PEALI RIVER [30]

ABSTRACT

The proposal of the Superintending Engineer, Southern Circle, about the re-excavation of a certain length of the upper reaches of the Peali River to a designed section and the effect on this of the tidal spill from the Matla River

[30] River Research Institute, West Bengal, Annual Report, 1947, pages 75-82.

was referred to the Institute for study in a scale model. The points that were asked to be investigated in the model were as follows :—

(1) If after the re-excavation of Peali River the connection between the Peali and the Matla is cut off by means of a *bund* across the Peali near Kultali what will be the effect of this construction on the regime of the Matla River. Provision for drainage and navigation of Peali will of course be made by means of a lock and sluice in the *bund* and a connecting channel with the Matla.

(2) If the Matla River is affected by the proposed *bund* across the Peali outfall what preventive measures should be taken to counteract this evil effect.

(3) What will be the effect of keeping the Peali River open to Matla spill even after the re-excavation. Can the Peali maintain itself with the limited drainage water supply ?

Results of experiments are given :—

The Matla is a tidal river and the tides from the sea face bring in large quantities of sand, silt and clay at every flow tide. These pass into the spill channels and ultimately into the spill areas. Twice during the day of 24-hours these sand, silt and clay are deposited during the periods of slack water on the beds of the parent river, spill channels and the spill areas. Both banks of the Peali being embanked all its spill areas have been cut off and it has only limited drainage during the rainy season. In consequence, its bed has been rising rapidly in the upper reaches and the drainage sluices have all become inoperative in this reach. If the upper reaches of the Peali are re-excavated to the designed section and left open to the spill of the Matla as suggested in the third alternative this reach will again silt up unless the following preventive measures are adopted at the same time :—

(*a*) Remove the embankment and allow the Peali to spill on both its banks.

(*b*) Bring in silt-free upland water supply all throughout the year to flush the channel.

Neither of these measures are possible now. The results of model experiments carried out by Prof. Reynolds in England are quoted in the institution of Engineers' Paper on "Deterioration of the Bidyadhari River" by Mr. D. N. Sen Gupta. These experiments of Prof. Reynolds had omitted to take into account the one most important factor which is peculiar to Bengal rivers in the tidal reaches, namely, the heavy silt and clay charge which these rivers carry in suspension. In consequence, these results of Prof. Reynolds cannot be applied to Bengal rivers.

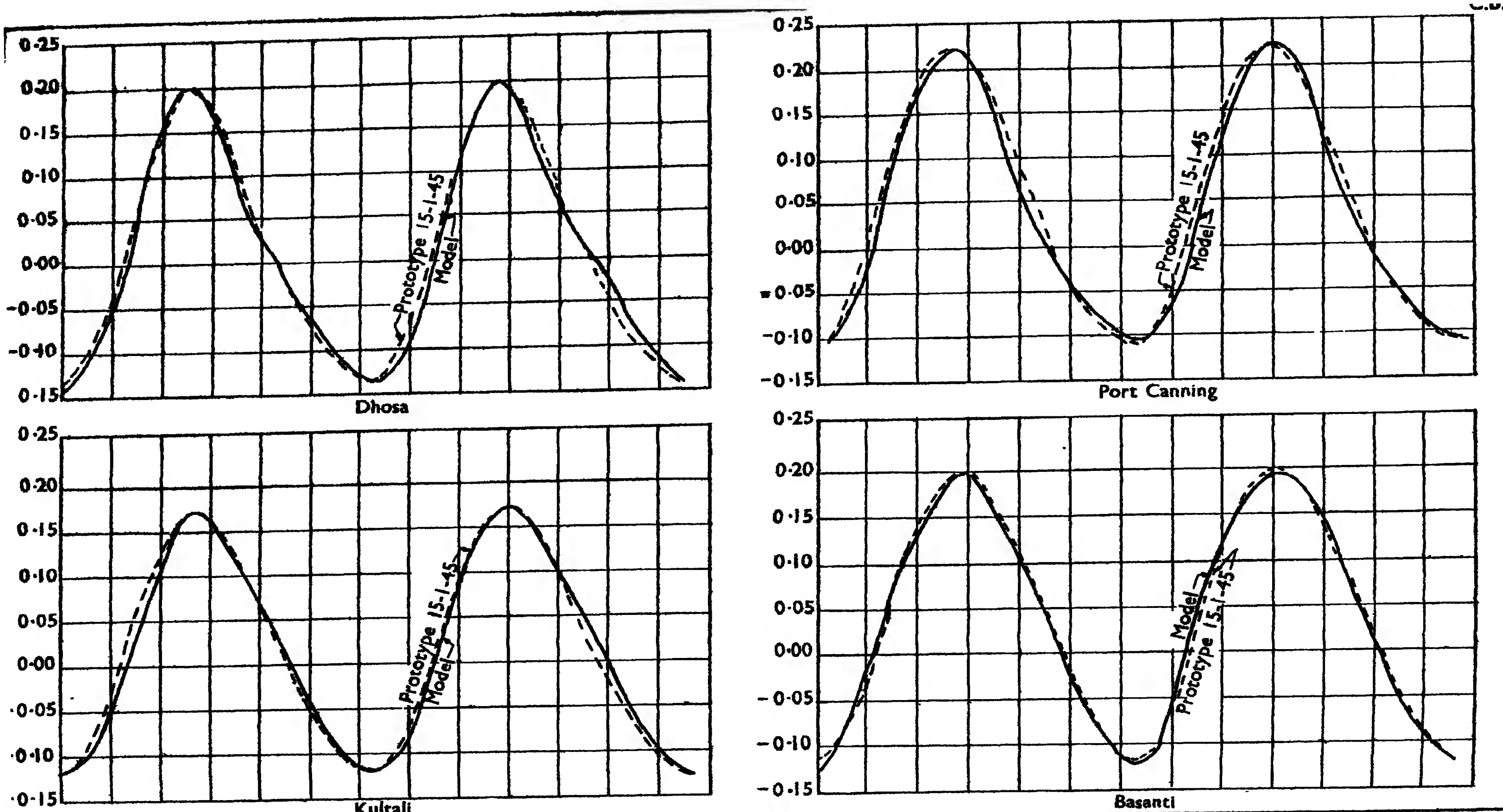

Figure 2C.22 (i) & 2C.22 (ii):-*Showing reproduction of tidal gauge in Matla Piali by the model*

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A model of Matla-Peali system was laid to a scale of $\frac{1}{500}$ horizontal and $\frac{1}{50}$ vertical. This included a stretch of the Matla River from Canning to a point about four miles downstream of Kultali, the outfall of the Peali and that of the Peali from its junction with the Bidyadhari to its outfall at Kultali. As the vertical scale distortion was high, the model bed and sides were laid in very weak cement. A cross-sectional survey of the River Matla for 1945 was available from the Port Commissioners of Calcutta who had also taken a set of simultaneous gauge readings of the Matla River at Canning and Basanti during a fortnight in January of the same year. A cross-sectional survey of the river Peali was made by the Executive Engineer, Canals Division, during 1945-46. No fresh bank survey of the Matla and Peali was undertaken. It was asserted by the Superintending Engineer, Southern Circle, that there had been no major changes in the river course since the cadastral survey of 1929 was completed. The model was, therefore, laid to 1929 survey with cross-section taken during 1945-46. The model was first "proved" from the gauge readings of the Port Commissioners for January 1945. The tides in this model were generated by a special device and the agreement between the observed and model gauge curves are shown in Figures 2 C. 22 (*i*) and (*ii*).

It had been pointed out in last year's annual report that the model gauges had been fairly reproduced at the downstream end at Kultali. Previously the gates were controlled by noting the level of water so as to reproduce the gauges near the gates. Afterwards a noteworthy change that was made in the operation of the gates for maintaining the gauge-curve was that the gate was operated by introducing a time-factor. By trial it was seen that the gauge readings of a particular tide could be reproduced by operating the gates at definite intervals of time and to definite marked positions of the gate (these positions being adjusted by trial and error) without noting the level of water. Introducing the time sequence into the operations, the gauges were seen to be better reproduced and cycles could be well repeated without much difficulty. Of course, for different tides the position of the gate markings required adjustments and alterations.

Another change that was made was this. It was first proposed that there would be two gates, one at the downstream end and the other at the upstream end of the model and the gauges would be reproduced by the operation of these two gates, water being fed from both the ends. But after running the model for some time, it was seen that the operation of the upstream end gate could be abandoned and this made the reproduction easier.

Hence as the supply water was not required during the ebb tide this was by-passed through another gate A, before entering the model, Figure 2 C. 23. This discharge was passed through a standing wave flume to a measure the volume passing out (Gate I and flume 2). The gate at the upstream end was, therefore, entirely blocked and the model was run only with one gate at the downstream

end (Gate II). The discharge passing out of this gate was passed through another standing wave flume III and this could always be measured (Figure 2 C. 23 shows the full arrangement).

One of the first signs of deterioration of a tidal river is the rise in its water level during high and low tides. The next sign to follow is the rise in the bed level. With the rise in the bed level the water levels rise still further till the bed rises so high that the tides can no longer flow over it and the river then dries up. This process has been followed by the Bidyadhari and this process was gradually creeping down the Peali up to Uttarbhag. In the model, therefore, the experiments were arranged in the following way. In the first series, the effect of closing the Peali by a *bund* on the maximum and minimum gauge readings in the Matla during flow and ebb tides were observed at a number of points in the river. The second series of observations dealt with the rise due to the closure of the Peali in the bed level of the Matla caused by bed movement and the third series with the rise in the bed level of the Matla caused by the settlement of suspended load. Series 1 and 2 have been completed and the results obtained so far are given here.

All the subsequent series of experiments in this connection were carried out under the following assumption. The validity of these experiments, therefore, depend on the validity of this assumption which is this. In a channel subject to tidal actions, the tidal impulses travel from the sea face up the channel and dissipate themselves gradually as they travel upstream. The magnitude and period of the impulses are determined by the tides in the ocean which in turn are governed by the attraction of the Sun and the Moon. It, therefore, comes to this that so long as the attraction of the Sun and the Moon remains the same and there is no change in the hydraulic characteristics of the estuary leading the tidal impulses from the sea face to the channel under consideration, the impulses travelling up to the channel will not be affected if any change is made in the channel itself. The impulses will come into the channel with the same intensity and sequence and will dissipate themselves in other parts of the channels not affected by the change. If, however, both the high and low water levels rise together it will indicate that there is not enough spill area to accommodate the surplus impulses from the channel. If there is sufficient spill area the high water level may rise slightly but the low water level will go down indicating that the surplus energy had been absorbed in the spill area and are released gradually during the ebb tide. This assumption had been utilised in the subsequent experiments.

A steady discharge was let into the model over a measuring flume (No. I) from the sump (Figure 2 C. 23). This was kept steady so long as the model ran. By manipulating gates No. 1 and No. 2, the discharge actually going into the model was regulated so as to reproduce one of the prototype gauge curves. In this case the gauge curve at Kultali was reproduced. In the prototype there are a number of spill areas connecting with the Matla. Their extent and levels are not known. Of these spill areas those near Basanti and Port Canning

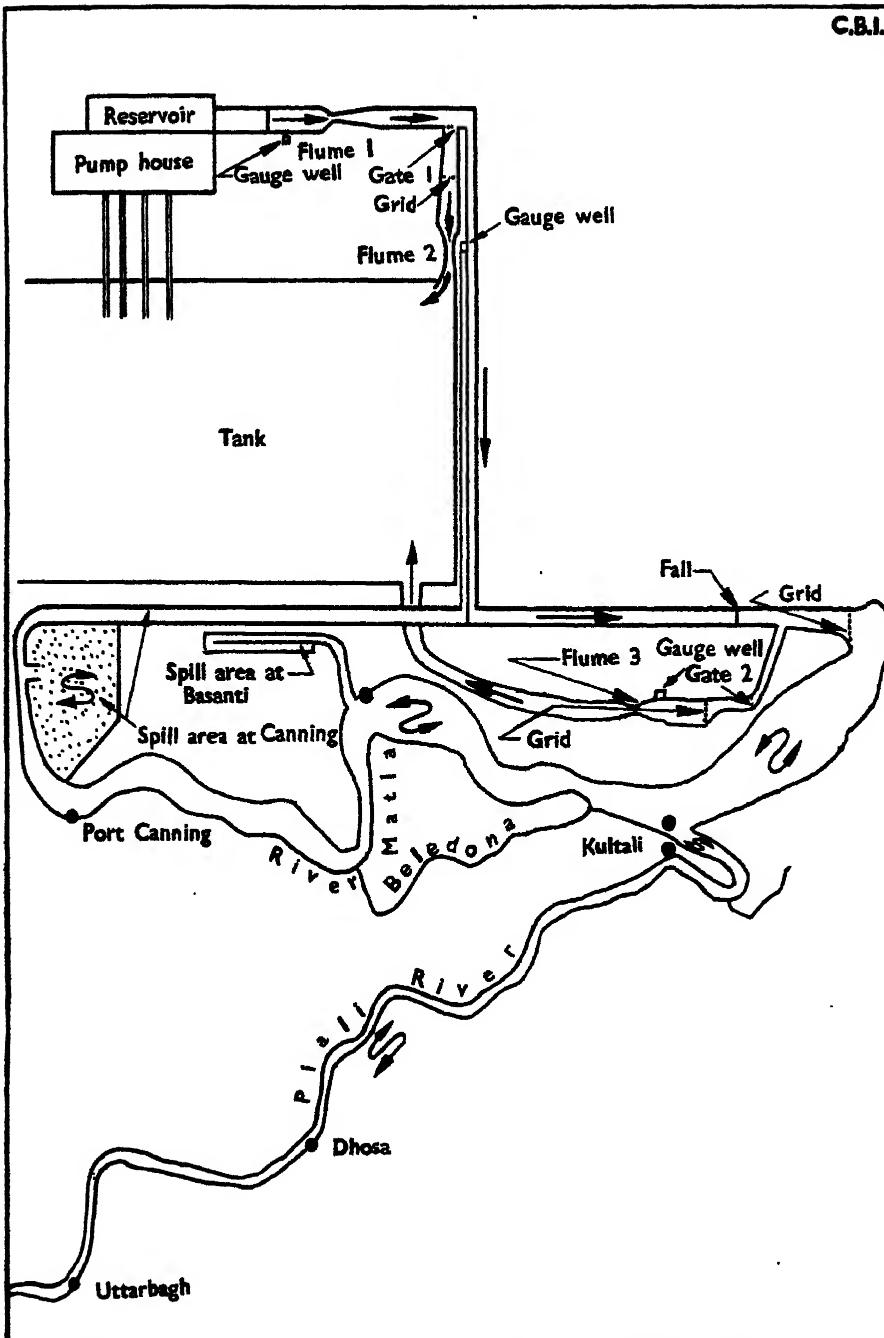

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Figure 2C.23 :- *Plan showing the arrangement at Matla Piali tidal model*

Scale 4 2 0 4 8 Miles

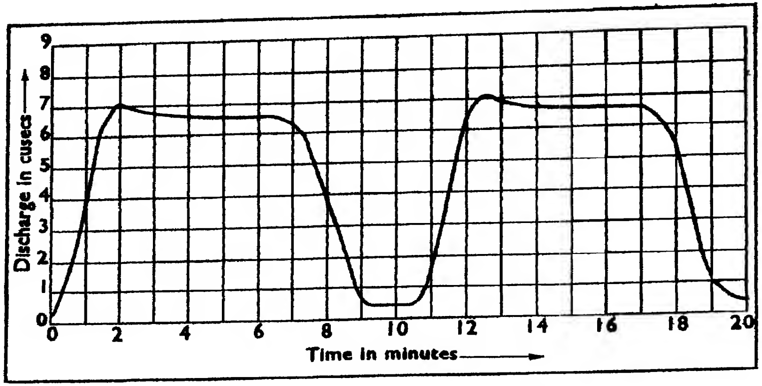

Figure 2C.24(i):-*Showing discharge in flume II before closure of Piali*

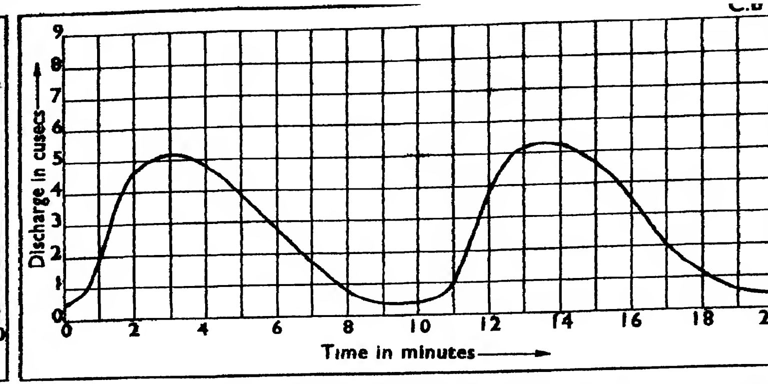

Figure 2C.24(iii):-*Showing discharge in flume III before closure of Piali*

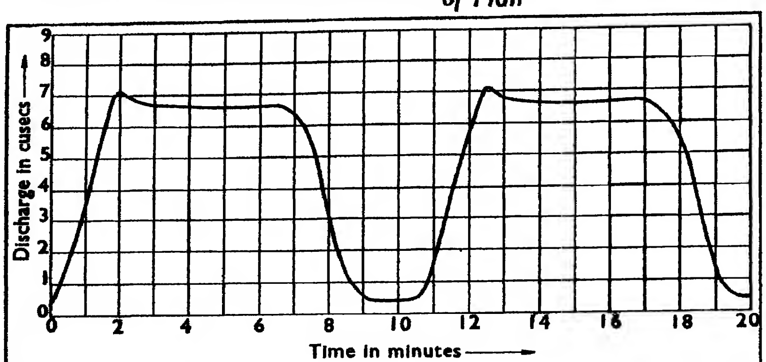

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Figure 2C.24(ii):-*Showing discharge in flume II after closure of Piali*

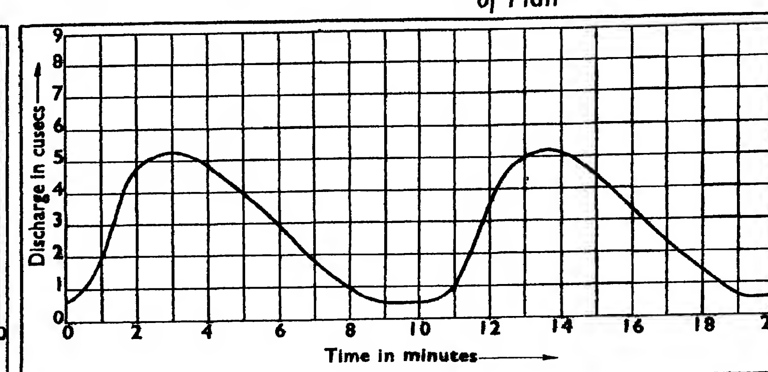

Figure 2C.24(iv):-*Showing discharge in flume III after closure of Piali*

are of any considerable size. There are also two gauges in the River Matla near these spill areas. The gauge curves for these two gauges are also known. In the model, therefore, after the Kultali gauge was reproduced by the manipulation of the gates 1 and 2, the spill areas of Basanti and Canning were adjusted to reproduce the simultaneous gauge courses at these two sites. When all the 3 gauges Kultali, Basanti and Canning were correctly reproduced the gauges in the Peali came out correct by themselves. This was possible as there is no spill area connected with the Peali which is embanked on both banks and does not allow any spill. The discharges passing down the flume 2 and 3 were measured and discharge curves plotted [Figures 2 C. 24 (*i*), (*ii*), (*iii*) and (*iv*]. While repeating the cycles efforts were made to reproduce these discharge curves faithfully so that one could be sure that the same sequence of discharge was passed into the model [Figures 2 C. 25 (*i*) and (*ii*)]. Great emphasis was laid on this aspect of the operation as otherwise there would be no certainty that the same intensity and sequence of tidal impulses are sent into the model for all the runs.

Series 1.—In this series after the model was proved, one hundred tides following the tide curve of March 7, 1947 at Kultali were reproduced in the model. The corresponding high and low water levels at Kultali, Basanti and Canning were recorded. These values were examined statistically and showed great consistency and very small standard deviation. The Peali was then bunded up at the point proposed and the same tide repeated one hundred times. The high and low gauges at Kultali, Basanti and Canning were again noted and analysed statistically. They showed the same order of consistency as previously. The average values of the high and low water levels of these two sets of observations were fundamentally different. The average value of the high water level after the closure was significantly higher than that before the closure, while the average low water level after the closure was significantly lower than that before. This indicated that though, after the closure, there was a likelihood of the high water level going up in the Matla giving indications of silting, the low water level would be lower in the latter case than in the former which showed that scouring would occur during the ebb improving the regime of the channel.

Further, surface velocities were observed by dropping floats in the Matla at the cross-section No. 42A during flow tides and at cross-section No. 4 during ebb tides before and after the closure. These are given in Tables 2 C. 12 and 2 C. 13. They show that the average values of the surface velocities increase in both cases after the closure. The total energies of flow across the River Matla both at Kultali, Basanti and Canning were calculated according to the following formula given by Dr. A. T. Doodson in Admiralty Manual of Tides (page 168).

$$\text{Total energy} = C \cdot g \rho p y^2$$

where c = a constant

y = rise in height above mean level.

The values obtained in the model were—

	Flow.	Ebb.
Total energy at Kultali—		
Before closure	2,178	2,178
After closure	2,450	2,450
Total energy at Basanti—		
Before closure	2,296	2,310
After closure	2,592	2,592
Total energy at Canning—		
Before closure	2,523	2.538
After closure	2,888	2,888

The total energy values after closure were in both cases higher than the pre-closure values indicating that there was no chance of silting of the Matla if the Peali is bunded up, Figure 2 C. 26 and 2 C. 27.

To obtain some direct evidence of silting in the Matla due to the closure of the Peali, experiments on series 2 and 3 were initiated. Silting in tidal rivers is caused by sand brought up from the sea end by tides rolling on the bed and also by silt and clay brought in suspension and dropped on the bed during periods of slack water. The two processes are entirely different and were, therefore, investigated separately in the model.

Series 2.—In this series " sand " (bed particle) in different varieties was injected into the model across a section of the Matla about three miles below the Kultali outfall of the Peali. Measured quantities of " sand " were injected from the moment the velocity started rising, that is, from immediately after low water and up to one quarter tide, for twenty tides consecutively during each run of one hundred tides. The exact kind of " sand " to be used in the model was determined by trial and error.

SAND

In the first experiment with sand as bed particles 0·5 cubic feet of sand was injected in the model just beyond the fall at the beginning of each of first ten cycles, the total quantity thus being 5 c. ft. In the subsequent experiments the total qunatity was raised to 10 c. ft. injected in 20 cycles just at the entrance of the model, that is down the cross-section No. 49A. In all, as many as seven experiments were conducted with sand as bed particles. It was observed that almost all the quantity of sand accumulated in the deep portion between cross-section Nos. 49A and 45A, very little movement being noticed beyond cross-section No. 40. The model was not generally run for more than 50 to 60 cycles a day, fresh quantity of sand being injected every day. Another experiment was continued (with 10 cubic feet of sand injected in first 20 cycles) for 400 tides without adding any more sand. These took a number of days so water was introduced into the model in the morning and taken out of the model at the end of the day very slowly, without disturbing the distribution of sand. There was movement of sand up to cross-section

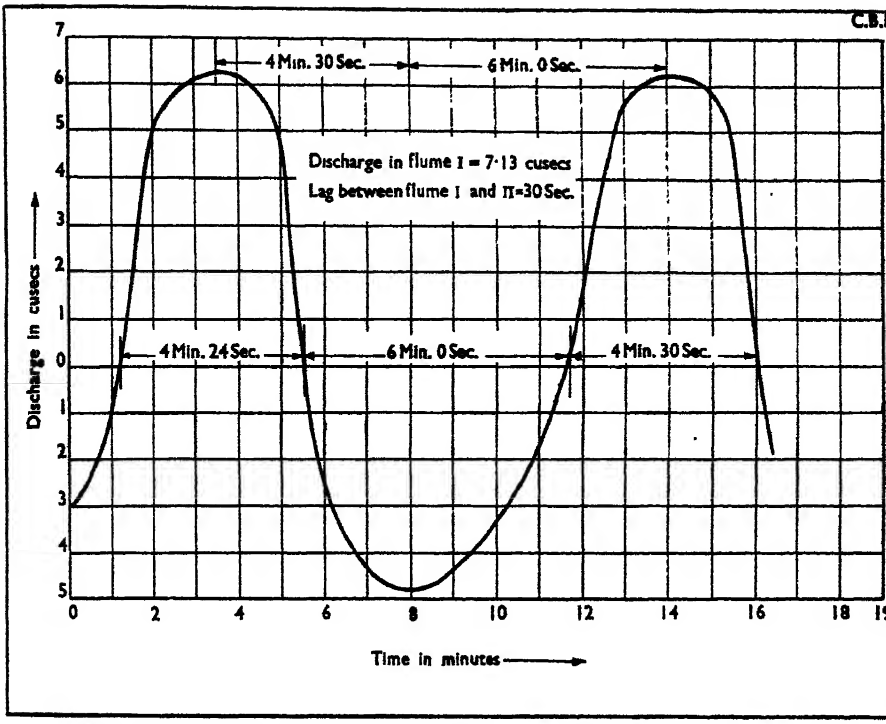

Figure 2C. 25(I):- *Showing discharge in the model before closure of Piali*

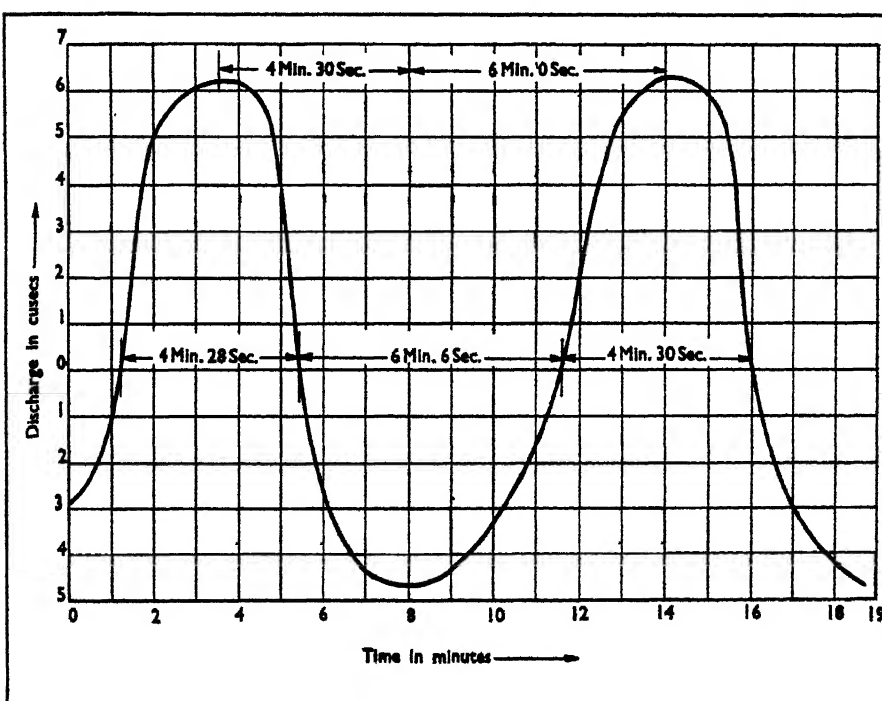

Figure 2C. 25(II):- *Showing discharge in the model after closure of Piali*

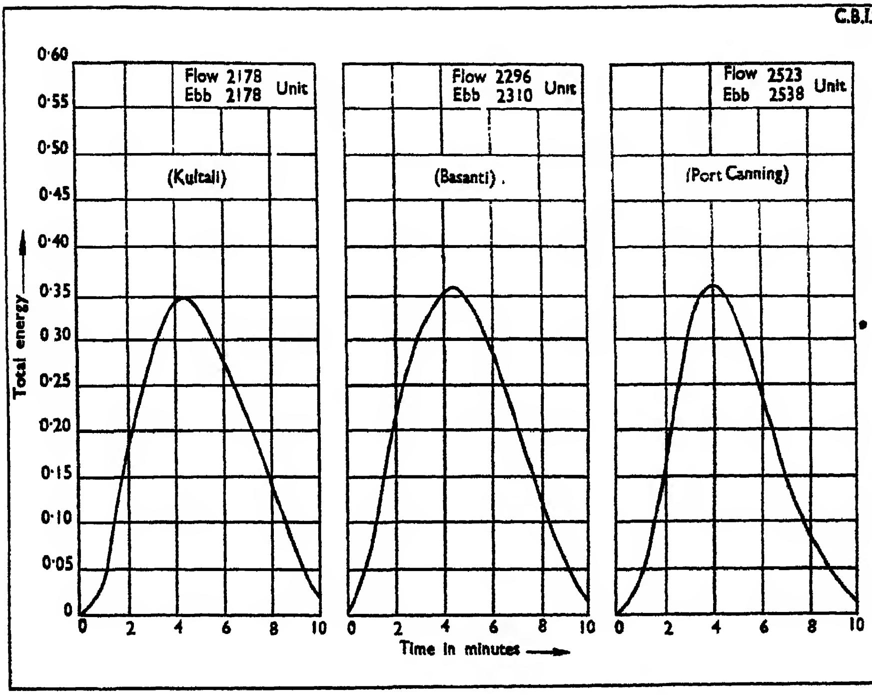

Figure2C.26:- *Showing energy calculations before closure*

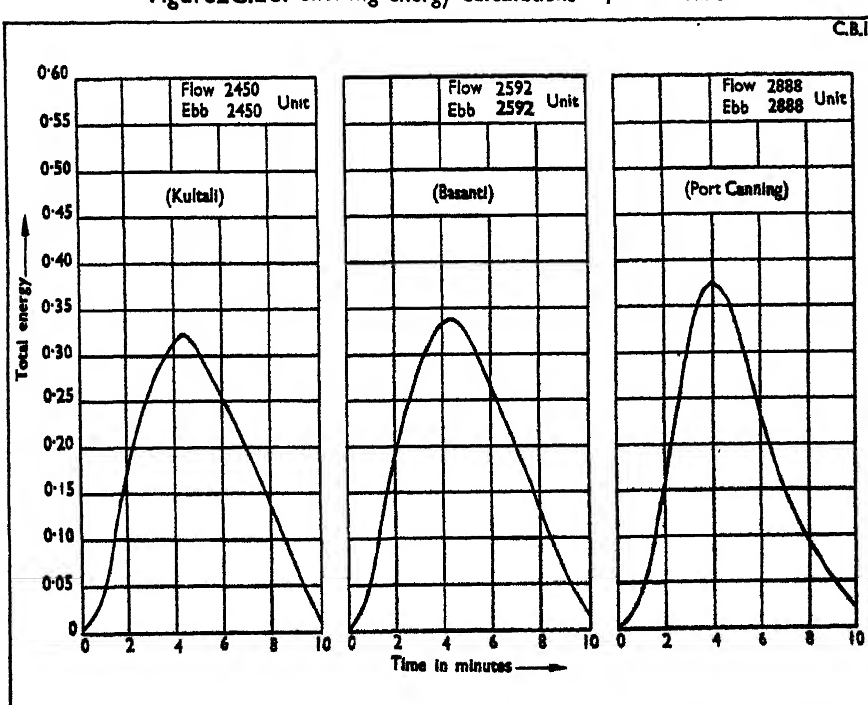

Figure2C.27:- *Showing energy calculations after closure*

Reg. No. 2737 XDD (C) '50-1,210 (P.Z.O.).

TABLE 2 C. 12

Observation of Float Velocity in the river Matla.

Flow tide.

Serial No.	Distance along the streamline in inches	Time in seconds	Velocity ft./sec.	Average velocity ft./sec.
	Before closure.			
1	1,940	255	0·63	..
2	1,972	256	0·64	..
3	2,075	270	0·64	0·64
4	1,762	223	0·66	..
5	1,762	230	0·63	..
	After closure.			
1	1,914	239	0·67	..
2	2,101	260	0·67	..
3	2,052	249	0·69	0·69
4	1,996	234	0·71	..
5	2,156	256	0·70	..

TABLE 2 C. 13

Observation of Float Velocity in the river Matla.

Ebb tide.

Serial No.	Distance along the streamline in inches	Time in seconds	Velocity ft./sec.	Average velocity ft./sec.
	Before closure on January 1, 1948			
1	1,940	320	0·53	..
2	1,762	270	0·54	..
3	1,979	309	0·53	0·53
4	1,876	302	0·52	..
5	1,762	285	0·52	..
	After closure on January 10, 1948			
1	2,133	323	0·55	..
2	2,111	310	0·57	..
3	2,101	315	0·55	0·56
4	1,940	282	0·57	..
5	1,940	288	0·56	..

TABLE 2 C. 14

Analysis of bed samples collected from the chars in the Peali between Dhosa and Peali Railway Station.

Serial No.	Registered No.	Description of the sample	Percentage above 0·06 mm. diameter and below 0·6 mm. Per cent.	Mean diameter
1	1 (Peali) ..	Sampled from the dry bed of Peali between Peali Railway Station and Uttarbagh (2 miles from Peali Railway Station).	7·5	..
2	2(a) (Dum Dum)	Sampled from the " char " formed two months past.	5·5	..
3	2(b) (Dum Dum)	Bed	6·5	..
4	3(a) (Kalaboro) ..	Sampled from ½ mile down to Kalaboro sluice (previous char) left bank.	8·0	..
5	3(b) (Kalaboro) ..	Sampled from ½ mile down of Kalaboro sluice (right bank new char) 1½ft. depth.	5·76	..
6	4 (Nishantala) ..	Sampled from newly formed " char " (right bank).	4·73	..
7	5(a) (Jaytala) ..	Sampled from the newly formed "char" at Jaytala left bank.	17·14	..
8	5(b) (Jaytala) ..	Sampled from the newly formed "char" at Jaytala left bank from 1½ ft. depth.	7·44	..
9	6 (Moutala) ..	Sampled from the bed	46·25	..
10	7 (Chandakhali)	Sampled from the newly formed "char" from the left bank near Chandakhali sluice about 1 mile from Dhosa just by the side of the "char" containing very fine sand and clay (from 1½ ft. depth).	41·20	0·97 mm.
11	8 (a) (Karampara)	Sampled from the newly formed "char" at Karampara (left bank) top surface.	86·0	0·124 mm.
12	8(b) (Karampara)	Sampled from the newly formed char at Karampara left bank (1½ ft. depth).	86·0	0·106 mm.
13	8(c) (Karampara)	Sampled from the newly formed char at Karampara right bank from 1½ ft. depth (1 furlong down to the point where only sandy "char" was found) 1 mile upstream of Dhosa.	10·8	..
14	8(d) (Karampara)	Sampled from the surface of newly formed "char" at Karampara right bank surface (1 furlong down to the point where only sandy " char " was found) 1 mile upstream of Dhosa.	Mostly clay.	..
15	9 (Dhosa) ..	Sampled from the "char"—Left bank at Dhosa sluice (1½ ft. depth).	72·28	0·088mm.
16	10(a) (Peali) ..	Between 24-25 miles	Nil	..
17	10(b) (Peali) ..	2 ft. depth between 24-25 miles ..	Nil	..

No. 35 and few particles of sand were seen on the elevated portions between cross-sections Nos. 30 and 25. To get better result the model was, therefore, run continuously for 300 tides without stop with the same quantity of sand introduced in the same way and the result noticed at the end of the experiment. It was seen that distribution had been better than discontinuous 400 tides but traces of particles could not be found beyond section 25.

Hence when after continuous 300 tides no appreciable movement of sand was noticed, it was concluded that with the velocity of water in the model and the friction of the model " bed sand " was not suitable for bed movement experiment. The next choice therefore was cinder.

CINDER

Boiler ash was washed and from the ashes small burnt coal particles were collected and dried. They were then broken up into small bits of different sizes (from powder to 3·5 mm.). Powder and fine particles were then sieved off as they move easily in suspension and large sized particles were used in the experiments (1 mm. to 3·5 mm.).

The quantity of cinder used in an experiment varied from 0·25 cubic feet to 4 c. ft. It was injected into the model in the first few cycles during the stagnation period between ebb and flow tides.

After about four or five cycles from the beginning small particles were found to be moving along the bed between cross-section No. 12 and cross-section No. 5. Large particles in considerable number were seen moving from deep points to deep points over elevated portions between them. These particles ultimately went into the spill beyond Port Canning. As the number of cycles went on increasing (from about 15th cycle) movement of larger particles became less appreciable and most of the cinder injected remained heaped up between cross-sections Nos. 49 and 45 and with still further increase in the number of the cycles (near about 40th cycle) there was practically no movement of particles in the bed of the river.

The experiment was repeated several times with the point of injection of cinder changed. But the same result always followed. Accumulation at the point of injection continued on every occasion and bed movement was only due to particles of low density. As it was not possible to separate burnt coal particles of this density in good quantity and as coal particles of low density (brown coal) are not available, experiment with cinder for bed movement was not continued.

MUSTARD SEEDS

The experiment was next tried with mustard seeds. Mustard seed has got a lower density than cinder and particles are also uniform. Definite quantity of mustard seeds (about 1 seer in 4 cycles in each experiment) was injected into the model at cross-section No. 49A in the same way and continuous cycles were run. This time the movement was very satisfactory in the Matla. During every flow tide the particles easily moved ahead, passed over elevated portions, from deep point to deep point and ultimately went over to the spill area beyond Canning. The particles did not accumulate at the point of injection and after about 40 cycles nearly 50 per cent. of the mustard seeds went into the spill area the remaining portion being distributed along the whole river. But though the movement in the Matla was good, particles entering the Peali were few in number. Moreover this entrance into the Peali was seen to depend upon the position and manner of injection. As the beginning of the model section at the downstream end (49A) was very near to the mouth of the Peali, the model was extended further 3 miles downstream (16 feet in the model scale) and the position of the gate II was also changed so that it might not have any effect on the movement.

The point of injection of the particles was moved further down and the manner of injection was standardised by dropping the particles from the same place in the same way and at the same instant (of a cycle). But the result was practically the same. Particles entering into the Peali by bed movement were not appreciable. Particles which entered into the Peali remained mostly within the loop (the quantity entering the Peali is not more than 1 per cent. of the total quantity injected). Those which could get out of the loop were seen to move along the bed very swiftly right up to Peali railway station. Particles were seen, after 40 cycles, to form groups between miles 12 and 11, miles 10 and 8, miles 7 and 4 and miles 3 and 0 but in none of the groups the number exceeded 1 per cent. of the total entry in Peali. No particles were seen to accumulate between miles 25 and 12. This clearly shows that the poor bed movement in Peali is not due to low velocity or high friction of the bed. To be sure, good quantity of mustard seeds was injected (about ½ seer) just at the mouth of Peali and the model was run continuously for 100 cycles. It was found that the number of particles escaping the loop is not more than 0·5 per cent. and the result as before followed.

The natural conclusion, therefore, that can be drawn from the above experiments is that there is little bed movement in Peali, and siltation in Peali is mainly due to suspended silt particles.

One inevitable conclusion from these runs that were repeated a number of times with various qualities of " sand " injected (about one seer) is that there is practically no bed sand movement in the Peali above its junction with Kultali. To verify this finding of the model, a number of bed samples were collected from the Peali river itself during low water of 5th February 1948. This survey was started from the Peali railway station and carried down to Dhusa up to

which a number of "chars" (islands) were seen during low ebb. Below Dhusa there was no "char" visible. Two samples were collected from a point between miles 24—25. All these samples have been analysed in the laboratory and the results given in the Table 2 C. 14. The percentage of sand particles between 0·05 to 0·6 mm. were obtained and analysed in the siltometer. The mean diameter for those samples that contained appreciable quantity of sand between these limits are also given in the Table. This analysis shows that expecting for samples collected near.about a mile upstream of Dhusa the "chars" contained fine clay which must have been carried by the flood water in suspension. This confirmed the findings of the model.

Series 2 has, therefore, shown that very inappreciable quantity of bed sand enters the Peali from the Matla. Hence the danger of a *bund* across the Peali stopping the entry of bed sand into the Peali from the Malta bed does not exist.

Experiments on series 3 are now in operation. These experiments will deal with the question of silting from the suspended particles carried by the flood water.

Experiments so far carried out indicate that no harmful effect is likely to accrue to the Matla if the Peali is bunded up at the point proposed.

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(3) MATLA PELALI TIDAL MODEL—ANALYSIS OF DATA [31]

ABSTRACT

The statistical analysis of the gauge readings of the Matla at different points is given here.

It was at first tested whether the tidal cycles as observed in the prototype was being reproduced in the model. For this purpose after proper manipulation of the gates and the spill area, the model was operated for more than 100 cycles, keeping the mouth of the Peali open, in order to reproduce the cycle as observed at Kultali. The gauge heights (maximum and minimum heights attained) and the time when these were obtained were recorded at three points in the model, viz., Kultali, Basanti and Port Canning.

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[31] River Research Institute, West Bengal, annual Report 1947, pages 83-85.

Investigations were at first made as to whether the high water levels and low water levels as recorded in the model were consistent in themselves or not, that is, as to whether the same level was more or less fairly well repeated in the cycles or not. The mean, standard deviation, and coefficient of variation (that is, the percentage that the standard deviation bears to the mean) for both high water level and low water level at each of the three points are tabulated in Table 2 C. 15. It is found from the table that the coefficient of variation was below size (that is, standard deviation about mean is less than six per cent. of the mean value), for high water level and below 10 for low water level at all the three stations. The standard deviations are thus reasonably small and the data might, therefore, be said to be fairly consistent.

TABLE 2 C. 15

Highest and lowest water levels of the Matla Model with the mouth of the Peali opened.

No.	Site	River stage	Number of observations	Mean level	Standard deviation	Coefficient of variation
1	Kultali	.. H. W. L.	108	0·2146	0·0102	4·7
		L. W. L.	106	—0·1292	0·0117	9·0
2	Basanti	.. H. W. L.	106	0·2513	0·0118	4·6
		L. W. L.	104	—0·1142	0·0103	8·9
3	Port Canning	.. H. W. L.	107	0·2573	0·0113	4·3
		L. W. L.	104	—0·1101	0·0065	5·9

The mean values of the high water level and the low water level at Kultali as recorded in the actual river, when converted to model scales, are 0·215 inch and —0·13 inch respectively, whereas in the model these values have been 0·2146 and —0·1297 respectively. The differences thus obtained are very small and when tested statistically are found to be not significant. The values of t,

$$t = \frac{\text{difference of means in prototype and model}}{\text{standard error of the model mean}},$$

are given in Table 2 C.16 with the five per cent. significance levels which are the values of t, the chance of exceeding which in a random sampling is five per cent. only.

TABLE 2 C.16

	Observed value of t	Degree of freedom	5 per cent. level of significance
High Water Level	0·41	107	1·98
Low Water Level	0·70	105	1·98

As the observed values of t are much below the five per cent. level of significance, the difference between model and prototype levels are not significant.

The model was then run for over 100 cycles with the mouth of the Peali closed and keeping all other things the same as before. The high water level and low water level as recorded now at the three stations were similarly tested for consistency. The mean, standard deviation and coefficient of variation are given in Table 2 C.17. In this case also the coefficient of variation is below 6 for high water level and below 12 for low water level at all the three stations. The standard deviation about the mean may therefore be said to be sufficiently small compared to the mean, and the data consistent in themselves.

TABLE 2 C.17

Highest and lowest water levels of the Matla Model with the Peali closed.

Serial No	Site	River stage	Number of observations	Mean level	Standard deviation	Coefficient of variation
1	Kultali ..	H. W. L.	112	0·2409	0·0114	4·73
		L. W. L.	112	—0·1152	0·0135	11·70
2	Basanti ..	H. W. L.	112	0·2821	0·0137	4·85
		L. W. L.	111	—0·1067	0·0118	11·10
3	Port Canning ..	H. W. L.	112	0·2883	0·0105	3·64
		L. W. L.	110	—0·1086	0·0080	7·41

It is found that after closure of the mouth of the Peali the high water level has on an average been raised by same amount. *viz.*, by 12 per cent. at all the three sites, *viz.*, Kultali, Basanti and Port Canning. The significance of these rises has been tested by Fisher's t, which is the difference between the values of the variates compared, scaled against an estimate of the standard deviation :—

$$t = \frac{\bar{x}_1 \sim \bar{x}_2}{\sqrt{\frac{(n_1-1)s_1^2+(n_2-1)s_2^2}{n_1+n_2-2}}} \div \sqrt{\frac{1}{n_1}+\frac{1}{n_2}},$$

degree of freedom $= n_1 + n_2 - 2$.

where x_1 and x_2 are the means, s_1 and s_2 are the standard deviations, and n_1 and n_2 are the number of observations of the variates compared. The calculated values of Fisher's t are given in Table 2 C.18.

TABLE 2 C.18

Site	Value of Fisher's t	Degrees of freedom	5 per cent. significance level	Significance
Kultali	17·81	218	1·97	Very high
Basanti	16·63	216	1·97	Ditto
Port Canning	20·84	217	1·97	Ditto

It is found from the above that the values of t obtained here are much higher than the five per cent. significance levels, so that the difference (that is, the rise) is very highly significant.

After extension of the model.—The model was extended at the lower reach and the spill areas adjusted in accordance with the revised gauge levels of the prototype. After the extension, similar reproduction of the tidal cycle in the model was attempted. Results of running the model for 100 cycles with the mouth of Peali open and for 100 cycles after closure of the Peali were statistically analysed in the same way. To test for the consistency of the data the coefficient of variation has been claculated for H.W.L. and L.W.L. at each of the points of observation, *viz.*, Kultali, Basanti, and Port Canning. Tables 2 C.19 and 2 C.20 give these coefficients of variation (percentage ratio of the mean to the standard deviation) for the model before and after the closure of the Peali.

It is seen from the tables that the coefficient of variation is below five for high water level and below 7 for the low water level at all the stations before the closure of Peali and below 4 for the high water level and below 6 for the low water level after the closure, which shows that the levels have been fairly repeated all the time.

The difference (that is the rise or fall) of the maximum (or minimum) levels before and after closure of Peali have been tabulated in Table 2 C.21. In this case it is found that there has been a significant rise in the high water level, the amount of rise being 7 per cent. at all the stations, while in the case of low water level there has been a fall by 8 per cent. at Kultali and six per cent. at Basanti, both the falls being highly significant, while at Port Canning the lower water level has been raised by a very small amount (not significant).

TABLE 2 C.19

Highest and lowest water levels before closing of the Peali in the extended model.

Serial No	Site	River stage	Number of observations	Mean level	Standard deviation	Coefficient of variation
1	Kultali ..	H. W. L.	100	0·1995	0·0079	3·96
		L. W. L.	100	—0·1227	0·0087	7·09
2	Basanti ..	H. W. L.	100	0·2178	0·0095	4·36
		L. W. L.	100	—0·1268	0·0065	5·13
3	Port Canning ..	H. W. L.	100	0·2600	0·0067	2·58
		L. W. L.	100	—0·1039	0·0044	4·23

TABLE 2 C.20

Highest and lowest water levels after closing of the Peali in the extended model.

Serial No	Site	River stage	Number of observation	Mean level	Standard deviation	Coefficient of variation
1	Kultali ..	H. W. L.	100	0·2133	0·0071	3·33
		L. W. L.	100	—0·1324	0·0077	5·82
2	Basanti ..	H. W. L.	100	0·2333	0·0066	2·83
		L. W. L.	100	—0·1343	0·0074	5·51
3	Port Canning ..	H. W. L.	100	0·2790	0·0066	2·37
		L. W. L.	100	—0·1030	0·0057	5·53

TABLE 2 C.21

Effect of closing the mouth of the Peali in the extended model.

Serial No	Site	River stage	Mean level (before closure) m_1	Mean level (after closure) m_2	Rise in level m_2-m_1	Percentage of rise $\frac{(m_2-m_1)}{m_2} \times 100$	Fisher's "t"	Degrees of freedom	Significance
1	Kultali ..	H. W. L.	0·1994	0·2133	0·0139	7	13·1	198	Highly significant
		L. W. W.	—0·1227	—0·1324	—0·0097	8	—8·4	198	"
2	Basanti ..	H. W. L.	0·2178	0·2333	0·0155	7	13·4	198	"
		L. W. L.	—0·1268	—0·1343	—0·0075	6	—7·6	198	"
3	Port Canning ..	H. W. L.	0·2600	0·2790	0·0190	7	20·0	198	"
		L. W. L.	—0·1039	—0·1030	0·0009	1	1·2	198	Not significant

DISCUSSION BY THE RESEARCH COMMITTEE

Introducing item (1) Dr. N. K. Bose said that he had considerable difficulty in assessing the silt entry into the spill channel Chandana. As the model was run during rainy season when the water of the Damodar canal was muddy a set of results were obtained which could not be reproduced later on when silt free water was received. This difficulty of carrying out silt experiments in muddy water might have been experienced by other workers who were requested to state their experience about it.

In introducing items (2) and (3) Shri P. B. Roy said that the object of the model study was to investigate the likely effect on the Matla River if the mouth of Peali was closed up by a lock and sluice.

For this purpose, the model was first proved by the correct reproduction of all the gauges by various adjustments. The model was then run for 100 tides with the mouth of Peali open and also closed with the same sequence of discharge in the model and the change in the high water and low waters was observed The results of the experiment with the extended model at the downstream end showed that though the high water went up at all places when Peali was closed, the low water went down. Of course the lowering diminished with the increase of the instance from the downstream and indicating that the ebb time was as if Peali was bunded up. The average scouring would increase during the value of the float velocity and the energy of flow and of ebb-tide also increased when Peali was closed.

As the problem was of siltation and siltation was due to (1) bed particles and (2) suspended particles, the bed movement was first taken up and studied. For bed particles sand was tried at first, but even though the model was run continuously for about 52 hours (for 300 tides) the bed movement was not satisfactory ; of course the velocity in the model was not sufficient for satisfactory sand movement. Cinder was tried next. In this case movement was satisfactory when the particles were of lower density. But as it was difficult to get cinder particles of appropriate density in a homogeneous mixture, this was also rejected. Mustard seed was next examined. In this case the movement was very satisfactory in Matla but there was little bed movement in Peali. Various adjustments were made for 'sand' entry into Peali but without any result. The bed particles were, therefore, injected at the mouth of Peali and the model was run for about 100 tides. It was noticed that most of the particles remained confined within the loop just beyond the mouth of Peali. But those particles which could get out of the loop moved swiftly along the bed indicating that the bed movement in Peali was not due to low velocity on higher friction of the bed. Samples from the bed of Peali were, therefore, collected and analysed. The analysis also confirmed the model findings that there would be no harm in Matla due to bed movement if Peali was closed up and the siltation in Peali was due to suspended silt. The study with suspended silt was in progress.

During discussion MR. GOVINDA RAO said that in the Matla—Peali model, its vertical exaggeration had been given in order to effect both dynamic and kinematic similarity. This model was distinguished from others in as much as a good lot of silt was brought by the sea also during its ebb and flow conditions. Since the rate of settling of the silt could not be made scalar, one would like to know how similitude was established in silt densities.

The President closed the discussion and said that the subject would remain on the Agenda.

DISCUSSION BY THE BOARD

THE SECRETARY said that three items were discussed at the Research Commitee meeting page 676. There was no resolution.

(v) MEANDERING

PRELIMINARY NOTE

Meandering is regarded as a phenomenon of great importance in training firstly because it is believed that knowledge of laws of meandering would be of great value in the prediction of river movements and secondly, because the location cf meanders is an important factor in the positioning of training works. This sub-head has been on the agenda of the Board for the past many years. The Board has been collecting data and a questoinnaire to that effect was issued in 1838. The digests of the replies received were compiled by C.C. Inglis [32], who also analysed data and established relationship, between meander length, meander width and discharge [33]. Mr. Inglis also did experimental work on the subject [34].

It is now understood that Sir Claude Inglis has included a chapter on the subject in his book 'River behaviour and Control' which he has written at the request of the Government of India and which is at present under publication by the Central Waterpower, Irrigation and Navigation Commission.

The following items were discussed at the 1947 Research Comnmittee Meeting :—

(1) Deterioration of the River Kultigong.

(2) Proposed bridge near Shahbad on the Ram Ganga River.

Recent Literature

(1) Inglis C. C.—Meanders and their bearing on river training—The Institution of Civil Engineers, Maritime and Waterways Engineering Division, London, 1947.

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[32] The Central Board of Irrigation. Annual Report (Technical), 1939-40, pages, 110-114

[33] Central Irrigation and Hydrodynamic Research Station, Poona, Technical Note No. 12, 1938.

[34] The Central Board of Irrigation, Annual Report (Technical) 1943, page 46.

THE YEAR'S WORK

DISCUSSION BY THE RESEARCH COMMITTEE

There was no contribution on this subject and hence no discussion took place.

DISCUSSION BY THE BOARD

THE SECRETARY said that there was no contribution and no discussion at the Research Committee meeting.

(vi) TIDAL ACTION INCLUDING THAT IN ESTUARIES

PRELIMINARY NOTE

This sub-head was introduced for the first time in 1945. The following items were discussed at the 1947 Research Committee meeting :

(1) Model experiments for the revival of Matla Peali System.

(2) Measures adopted to reduce turbulence in the Hooghly river downstream of the knuckle at the Titaghar Jute Mill (Bengal).

(3) Protecting the foreshore of the Anglo India Jute Mill on the Hooghly river (Bengal).

Recent Literature

(1) Allen J.—Tidal model of the Firth of Tay—Engineering, Vol. 164, No. 425, September 15, 1947.

(2) Van Veen J.—Research of tidal rivers in the Netherlands—A successful combination of theory and practice—Dock and Harbour Authoriy, Vol. 27, Nos. 313 and 314, November and December 1946.

THE YEAR'S WORK

The following items were discussed at the 1948 Research Committee meeting :—

(1) Measures adopted to reduce turbulence in the Hooghli river down stream of the knuckle at the Titaghar Jute Mill.

(2) Preliminary note on the Cochin Harbour Model.

(1) MEASURES ADOPTED TO REDUCE TURBULENCE IN THE HOOGHLY RIVER DOWNSTREAM OF THE KNUCKLE AT THE TITAGHAR JUTE MILL [35]

ABSTRACT

This note describes the experiments carried out with the $\frac{1}{300} : \frac{1}{60}$ scale model and the $\frac{1}{150} : \frac{1}{80}$ part-length model of the Hooghly to test the effects on the flow conditions at the knuckle at the Titaghar Jute Mill, of—

(*a*) a series of Islands, and a (*b*) continuous sprating wall.

[35] Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report, Technical, 1947, pages 83—89.

The experiments showed that the conditions are so *set* that no practicable solution at reasonable cost appears feasible.

1. EXPERIMENTS WITH ISLANDS IN $\frac{1}{300} : \frac{1}{60}$

Introduction.—

Figure 2 C.28 shows the model reaches of the $\frac{1}{300} : \frac{1}{60}$ model and $\frac{1}{150} : \frac{1}{180}$ model of the river Hooghly. In the case of the knuckle at Titaghar Jute Mill, the " change-over " of the flow occurs immediately upstream of the knuckle leading to more turbulence downstream and in addition to this, there is a

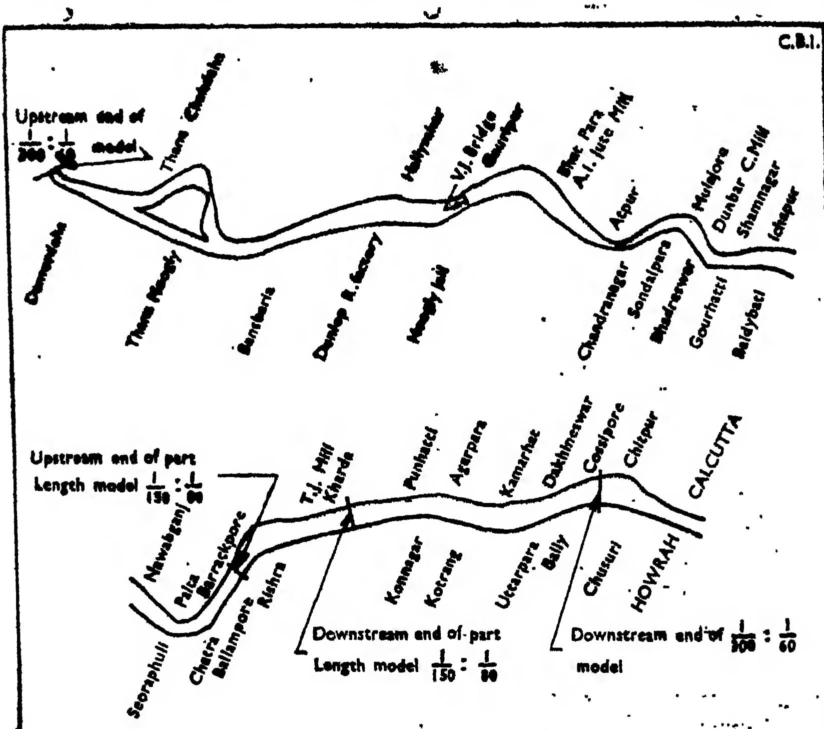

Figure 2C.28:—*Showing the model reaches of the River Hooghly*

throttling of the width of the river channel immediately downstream of the knuckle. These peculiar conditions made the action of a row of piles found successful in the case of the Dunbar Cotton Mill knuckle ([36]) *ineffective* at Titaghar in reducing the turbulence and inducing silting in the bight.

([36]) Central Waterways Experiment Station Poona, Annual Report, Technical, 1946 page 40.

The previous experiments, [37] with islands had been done with the object of getting maximum improvement of flow downstream of the knuckle with the least number of islands. It had been found that a single island 'B' ¾ mile upstream of the knuckle, attracted the flow on to the right bank, and no island upstream of 'B' could be effective in assisting the action of this island. It was, therefore, decided to try a series of islands, downstream of island 'B', for attracting flow to the right face of the islands and thereby relieving the concentration of flow at the knuckle.

The model was laid to the 1939-40 survey and freshet spring tide curves as observed in the river on September 23, 1941 were reproduced and repeated ten times; in each experiment '*V*' sand ($m = 0.20$ mm.) was used as bed material. Lines of flow were observed (after completion of 10 tides) under the worst condition of ebb tide when the water level at Konnagar was 13·5 feet above K. O. D. S. (Kidderepoer Old Dock Sill) and the discharge was × 187,000 cusecs. Besides, these observations could not obviously be made under tidal conditions.

The main requirement of these experiments was to reproduce freshet spring tides as observed in the river on September 23, 1941. For the correct reproduction of these tides three recorders carrying the model tidal curves were fixed at the upstream (*i.e.* Damurdah) and downstream (*i.e.* Cossipore) ends of the model and at Konnagar. The observations made in the experiment consisted of lines of flow and velocities at cross-section No. 6 under conditions mentioned above.

Various positions and alignments of small islands were tried to see if the concentration of flow could be reduced. The results, however, were not satisfactory.

A submerged island was constructed in the straight reach upstream of Palta, its top being kept five feet below K. O. D. S. thus allowing 10 feet draft. It was observed that such an island had little or no action and further experimental data were, therefore, not collected.

In this case, therefore, no single island or a series of islands could be evolved which would have sufficient action against the unfavourable combination of existing conditions.

2. EXPERIMENTS WITH A CONTINUOUS WALL ON THE SHOAL OPPOSITE THE KNUCKLE.

A continuous wall in line with the island was construcíed with a view to preventing flow occurring between two islands because such flow made the attack on the left bank severe during the last of ebb.

The top of the wall was kept = 25 feet above K. O. D. S., *i.e.*, four feet above high water level, so as to remain above high floods. The right side of the wall was given a slope equivalent to 1 in 3 in prototype while the left side facing the knuckle was given a slope equivalent to 1 in 5, with a view to induce more

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[37] Central Waterways Experiment Station, Poona, Annual Report, Technical, 1945.

scour in the right compartment by the steeper slope. The flat slopes would also enable the wall being constructed out of dredged bed material in the vicinity. The alignment of the wall was decided from initial experiments, the first care being to see that it would not hamper navigation.

The construction of the continuous wall not only stopped the flow between islands but it increased the discharge on the shoal.

The increase in discharge on right side appears to be the result of the curvature induced by the alignment of the wall. No more curvature could be given to the wall with advantage as the flow did not follow sharper curvature, leaving an intermediate region to slack water near the wall, as is clear by comparing Figures 2 C.29 and 2 C.30.

The more the wall is moved upstream and the wider the entrance on the right side, the more the discharge that is likely to be drawn towards the right bank; but both these factors are limited by the existing navigable channel.

It will be evident that the aim of all these experiments was to reduce the left bank discharge and create conditions favourable for silting downstream of the knuckle. The optimum alignment of the wall as in Figure 2 C.30 was, therefore, retested in a large, part-length $\frac{1}{150} : \frac{1}{80}$ model, to study the effects in greater detail.

Assuming the wall to be constructed out of the dredged material with slopes of 1 in 5 and 1 in 3, the percentage obstruction will be 7·6 and 6·7 respectively.

The effect on the general regime of the river would be negligible.

3. EXPERIMENTS IN $\frac{1}{150} : \frac{1}{80}$ MODEL

The model was laid with Londha sand (m = 0·30 mms.) to the 1939-40 survey. Discharge of 1·7 cusecs ≡ 187,000 cusecs was run continuously for 24 hours (× 16 days) in all experiments. Water level at cross-section 4A was maintained corresponding to W. L. at Konnagar ≡ 13·5 feet above K. O. D. S. The bed was adjusted at the beginning of each experiment to enable fair comparison. Continuous ebb was run to expedite results, as almost all the effect was observed to occur during ebbs; this was done after careful comparison of the effects with those obtained with tidal flow.

Experiments in $\frac{1}{150} : \frac{1}{80}$ model showed that with the construction of the continuous wall deposition of the silt downstream of the knuckle was possible. Construction of piles near the knuckle in addition to the wall, were found to help the action of the wall and accelerate silting. In all the experiments with the wall when injection of red silt was done, it was found that there was a tendency of silting on the left side of the wall facing the knuckle. It was, therefore, necessary to see whether this part near the wall remains silted under tidal action of two-way flow.

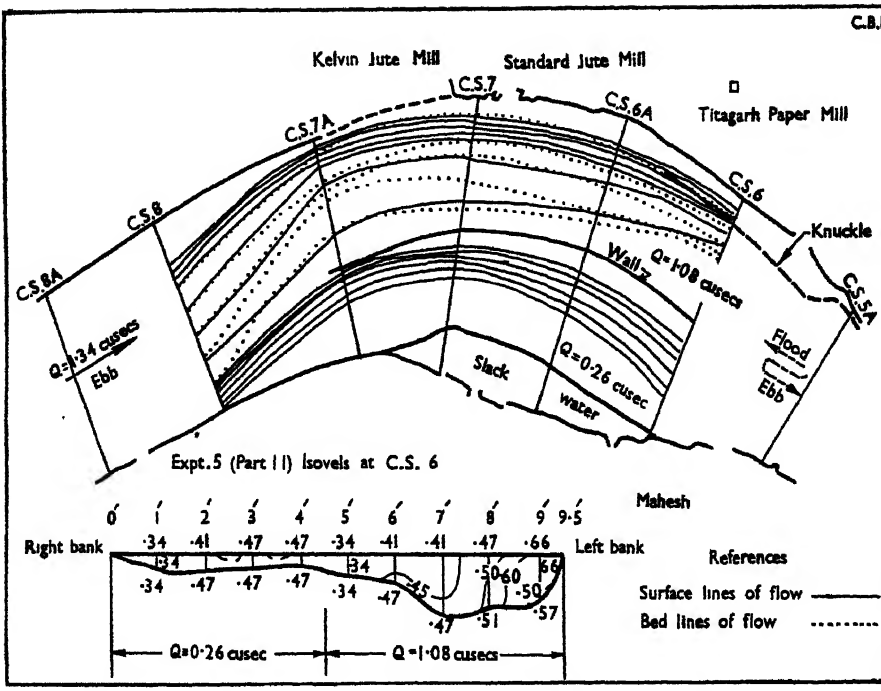

Figure 2C.29:- *Showing lines of flow with wall parallel to left bank*

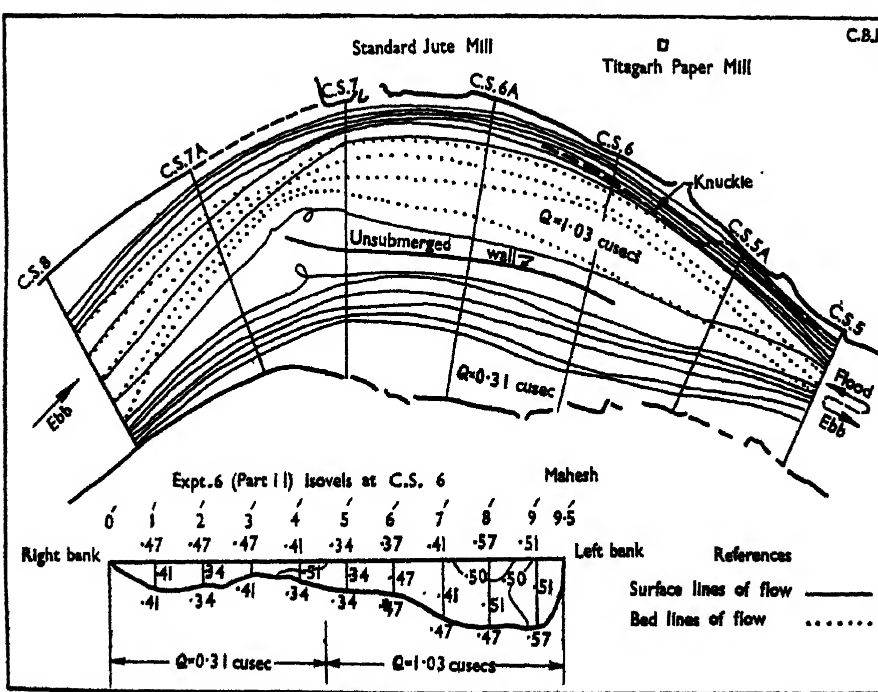

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Figure 2C.30 :- *Showing lines of flow with the nose of wall shifted downstream and moved away from the right bank*

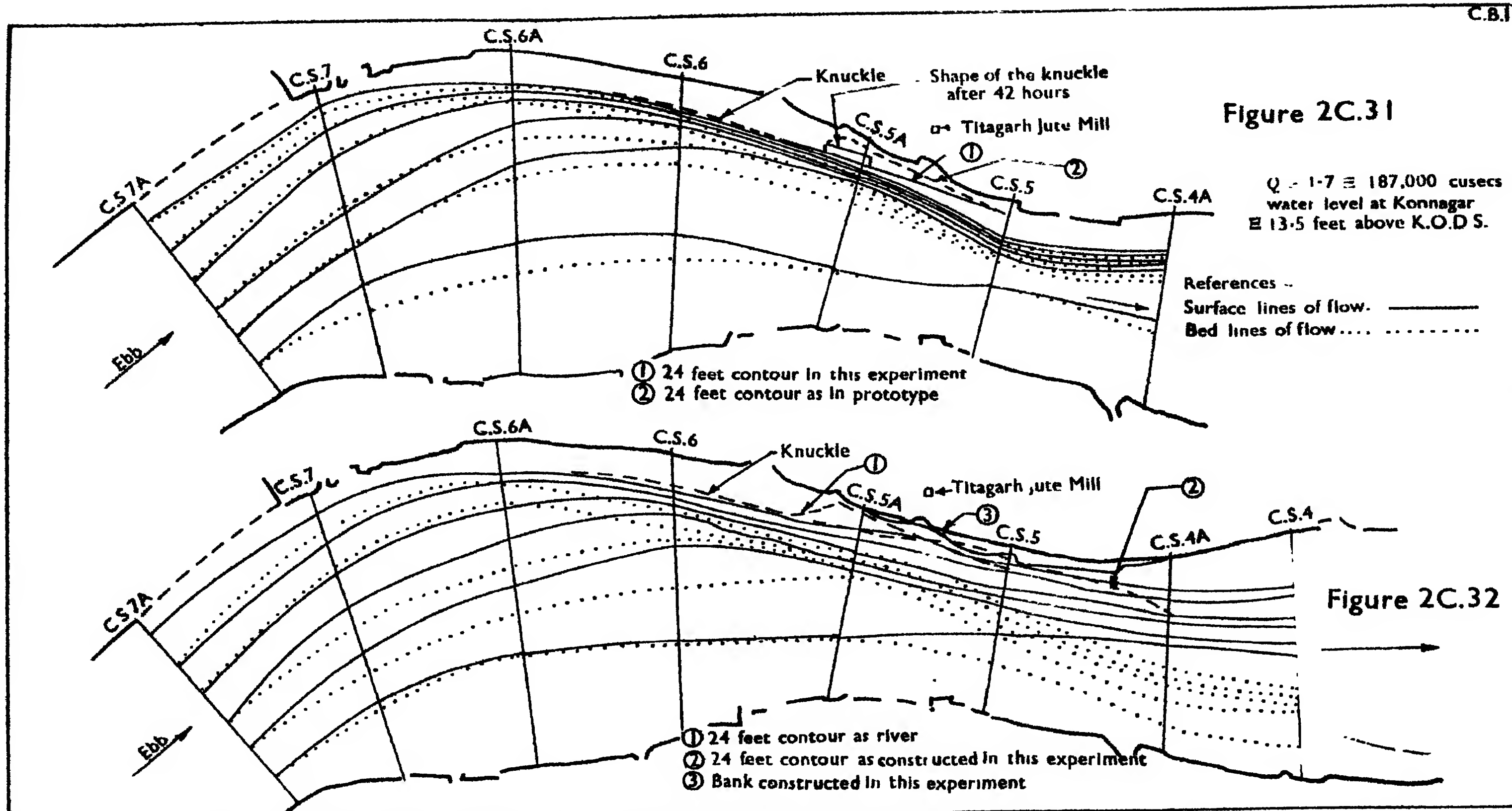

Figure 2C.31 & 2C.32:- *Showing lines of flow after 42 hours of continuous ebb* ($\frac{1}{50}:\frac{1}{180}$ *Hoogly River model*)

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In these experiments the effect of the wall was mainly tested under tidal conditions as against under continuous ebb in the previous series. The tidal curves for the upstream and downstream end of the model and at the knuckle were obtained from the simultaneous curves observed in the river on September 23, 1941 and in all experiments these curves were repeated 10 times before taking observations.

Correct bed movement cannot be expected in a model of so low a vertical exaggeration as $\frac{1}{150}:\frac{1}{80}$ which is unnatural for reproducing curvature; slight difference in flow conditions and general bed movement observed in the model, by comparison with the conditions obtained in the $\frac{1}{300}:\frac{1}{60}$ model were eliminated by small adjustment in the tidal curves at entry and exit (keeping the conditions and tidal curve in the observation reach near the knuckle the same). As the observation and change in water levels at the knuckle caused by the wall is insignificant, it is reasonable to assume that the same adjustments of tidal curves at entry and exist are sufficient and necessary with as without the wall.

These experiments showed that after accretion the conditions would again deteriorate, resulting in concentration of flow near the knuckle following the shoaling along the left side of the wall which is likely to occur. Thus, though initially the wall may induce some silting below the knuckle, this is likely to be of little lasting good. The conditions at the knuckle are so set that no practicable solution at reasonable cost appears feasible.

As a purely basic experiment, it was decided to test whether the contraction of the river section following the curve was primarily responsible for the formation of the knuckle.

The 24 feet contour of the knuckle was continued up to cross-section No. 5 where the contraction has taken place, to represent the bank under the pre-knuckle condition—*vide* Figure 2 C.31. The bank in the model, five feet upstream and five feet downstream of the knuckle, was constructed in alternating layers of sand and clay similar to the Mulajore borings.

Figure 2 C.3[illegible] shows the lines of flow under continuous ebb when the experiment was started showing the concentration of flow at the knuckle site.

After 12 hours of continuous ebb, a slip of the bank occurred at the site of the knuckle due to undermining of the sand layers. The material of the slip was removed, as the velocities in model were not sufficient to remove it.

After 42 hours of continuous ebb a shape of the bank similar to actual knuckle was formed showing clearly the tendency of the bank to cave at the knuckle site.

The object of this experiment was to see whether the knuckle at cross-section No. 5A is reformed if the existing contraction downstream of the knuckle is removed. The left bank was, therefore, cut back, as shown in Figure 2 C.32, keeping the same bank slope as before.

To get the desired width at cross-section No. 5, the width of the channel was found out from the Lacey formula $W = 2 \cdot 67 \sqrt{Q} = 2 \cdot 67 \sqrt{187{,}000} = 1160$ feet, as a first approximation, where W is the width of the channel and Q the maximum discharge.

The existing width of the channel is 900 feet and so the bank was receded by 260 feet at cross-section No. 5 to get the necessary width of the channel.

Continuous ebb flow was then reproduced. It was observed that there was much less concentration of flow at cross-section No. 5A the knuckle site—compared to that in the previous experiment. The experiment was continued for 42 hours and no bank slips were observed during this period.

It may be, therefore, concluded that the 'bottle-neck' downstream of the present knuckle has contributed towards the formation of the knuckle.

The experiments indicated that no simple remedy is possible to ameliorate the conditions at the Titaghar mill; drastic measures such as the removal of the 'bottle-neck' contraction downstream of the wall are not practicable, having regard to the small likely good and the not inconsiderable expenditure.

(2) PRELIMINARY NOTE ON THE COCHIN HARBOUR MODEL(38)

ABSTRACT

Gives a Summary of the problem and the method of selection of scales for the model

One of the strong reasons advanced in the early days (in 1880) for opening up the Cochin Harbour was that dredging maintenance would be light. Being a natural harbour with a copious back water (extending over 100 miles N—S in length 1½ to 15 miles broad, and of great depth) into which some considerable rivers from the distant Travancore and Cochin hill debouch, it was considered that the silt would be wholly deposited in the backwater reservoir so that, even in the height of the monsoon, the water flowing out of the principal passage into the sea at Cochin would be practically free of silt, precluding the formation of a bar. Later Silt observations made in the Cochin gut showed that more silt was brought into the harbour during the monsoon than was discharged.

In 1927-29, a sea approach channel to the Port, 16,000 feet long (of which 4,000 feet was on the east or landward side of the bar) and 450 feet wide, was dredged to a depth of 37 feet at Low Water Ordinary Spring Tide, starting from the 32 feet contour at the western and to the same contour at the eastern end of the bar. In the inner harbour, the Mattancherry Channel, mooring areas and the Ernakulam Channel were dredged.

(38) Summarised from Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report Technical, 1947, pages 91-113.

There has been, since 1933, much greater silting. As there were two very low annual rainfalls and one low one, all in successions, since 1933, it was thought at first that this low rainfall reduced the scouring effect of the monsoon freshet, which are usually regarded as the principal force in keeping the channel clear from the mud which the combined sea forces (the swell, the flood-tide *etc.*) push into the channel. It has been observed, however, that the entrance channel silts up as much as nine feet during the monsoon. When, despite twice the normal rainfall in July 1937 there was more than average silting in the approach channel—this was, in fact, the year of maximum silting—it became necessary to discover the cause of this heavy silting so that the Port may know what to expect in the future.

It was believed that the two most important contributory causes were (1) a known and measured movement Southward of the Narakal "mud-bank" entering the approach channel and causing inordinate silting ; and (2) an observed rising of the sea bed to the North of the harbour since 1934, which may or may not be a partial swelling and lifting of the bank, or a movement to the east of the sea bed to the north-west of the approach channel.

With regard to (1), it has been noticed that though the prevailing drift for sand and shells is unquestionably to the north east and that the coast line north of Vypeen was strengthened and widened by the hard material pumped up from the channel, the indisputable evidence of the resultant movement of the Narakal mud bank southwards shows that another force operates from the north sufficient to move the mud *but not sand.* (The southern fringe of the mud bank was found, by actual survey on 20th August, 1937, about a mile south of the Cochin entrance while it had been definitely north of it before).

A special committee was, therefore, appointed in 1938, charged with the following Terms of Reference :—

(*i*) What is the force of the combination of forces, which starts the Narakal mud bank moving north or south, and continues it in that movement after the original cause has apparently disappeared ?

(*ii*) Is there evidence enough to justify an opinion being given as to the cause of the rising of the mud bottom originating north of Cochin Harbour in 1934 ? Could it have been caused by the Bihar earthquake of 15 January, 1934 ?

(*iii*) Why is there not a similar rising originating south of the harbour mouth ?

(*iv*) Can it be stated, from the answers to the above, and from other evidence available, what was the immediate cause of the unusual silting in the Cochin channel in July 1937 ?

(*v*) Can any suggestion be made as to the possibility of keeping the mud bank in its present position, or near it, by a careful programme of dredging operations, maintenance or otherwise ?

Various theories in explanation of the silting in the Approach Channel and in the inner harbour are discussed in the " Blue Books " by Sir Robert Bristow on the History of Malabar Mud Banks and in the Expert Committee's Report.

The present position regarding silting and maintenance dredging is :—

" The Mattancherry channel was dredged to a width of 1,000 feet and a turning basin opposite the wharf to a width of 1,400 feet. The width of the turning basin was reduced about four years ago to 1,200 feet as siltation on the western side was very rapid, necessitating dredging three times a year. The silting which takes place in the approach channel amounts to between $1\frac{1}{2}$ and 2 millions c. yds. per annum and that in the inner harbour has about $1\frac{1}{2}$ millions c. yds.

The maintenance dredging of the approach channel is done by the Suction Dredger " Lord Willingdon " and pipeline in a period of about 4 weeks, in January-February, the most favourable months for pipe-line dredging in the sea. The silt which is *fine mud and clay* is pumped through a pipeline 3,000 feet long and deposited in the sea at a distance of approximately 2,000 feet south of the approach channel. It costs about Rs. 2 *lakhs* a year to do this dredging.

The maintenance dredging in the inner channel is done by the bucket dredger " Lady Willingdon " and three attendant hopper barges. The silt dealt with here also is *fine mud and clay* which takes a very long time to settle in the hoppers. Weights of three samples taken from the Mattancherry channel when the design of the hopper barges was being considered were 73·8 lbs., 75·5 lbs., and 83·5 lbs., per c. ft. In order to obtain as concentrated a hopper load as possible in depositing the spoil at sea the hopper doors are not dropped as is the usual practice. The hopper doors are kept sealed and the material is pumped out of the hoppers. The dumping ground at sea is about two miles south of the approach channel and about two miles west of the shore where the spoil is deposited in 30 feet depth of water. It takes about eight months in the year to do this maintenance dredging and the cost is about eight *lakhs* of rupees. The ten *lakhs* of rupees spent annually on inner and outer channel maintenance dredging absorbs an unduly large proportion of the total revenue of the Port which is approximately 36 *lakhs* of rupees.

The **areas** in the inner harbour where regular maintenance dredging is **required are :—**

(*a*) A width of 250 feet at the wharf frontage where dredging is necessary at least twice a year and often three times.

(*b*) The western half of the turning basin necessitating dredging twice a year.

(*c*) The rest of the area forming the turning basin which has to be dredged at least once a year.

(*d*) The mooring areas on the western side of the Mattancherry channel where dredging is necessary at least once a year.

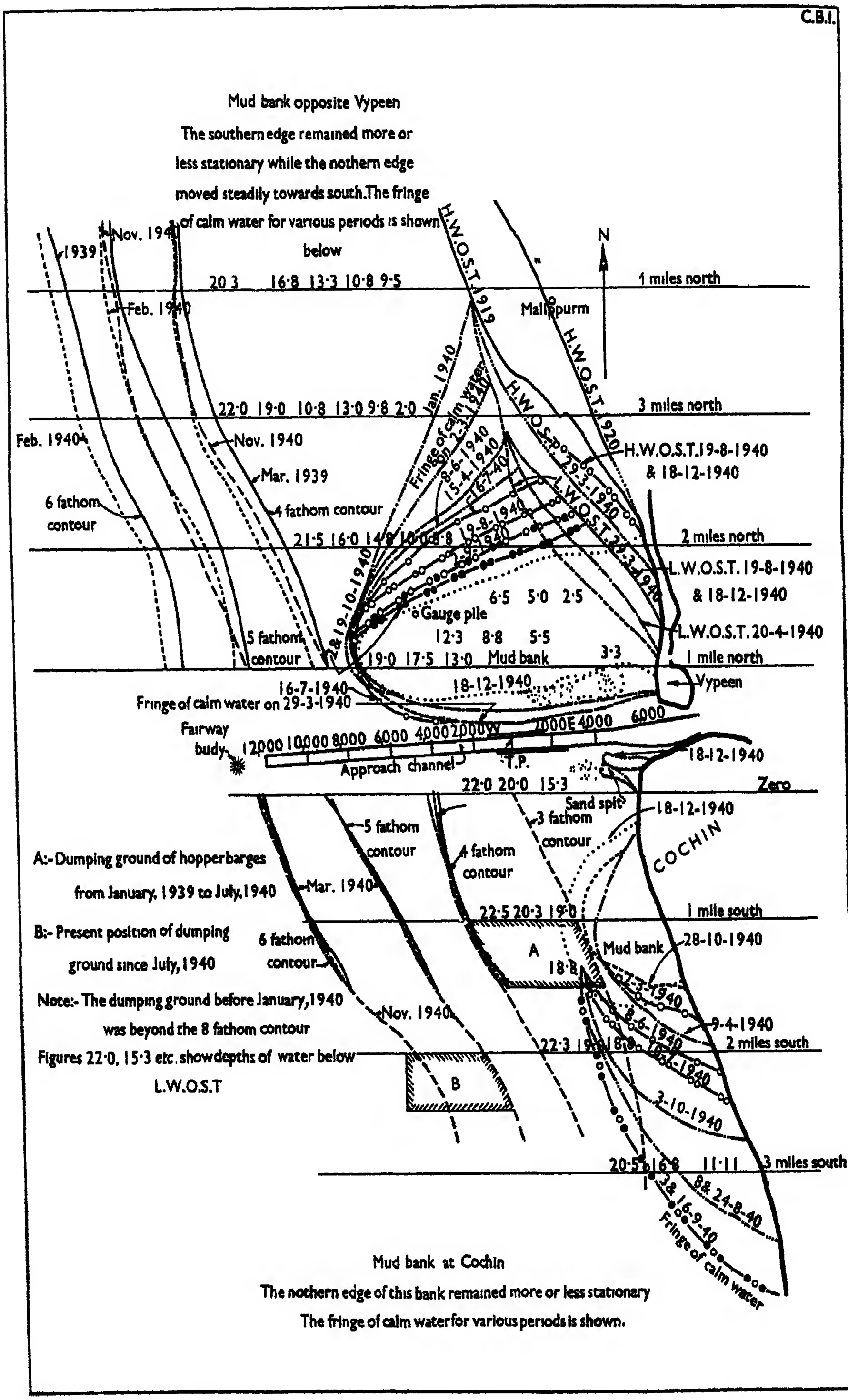

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Figure 2C.33:- *Showing Cochin port contours and mud banks*

Scale 4000 2000 0 4000 8000 12000 Feet

The silting takes place very largely in the monsoon months and although it remains to be proved conclusively, evidence and observation strongly favours the conclusion that the silt comes mainly if not wholly from outside harbour.

Silt observations made in recent years confirm this.

There can be little doubt that the fine silt in the sea bed around the approaches to the Port being easily stirred up and set in motion in the monsoon conditions is carried in suspension on the flood tide inside the harbour and deposited in quantity. It also seems a reasonable surmise that the bar which existed before the Port was opened up excluded a large proportion of the silt which finds easy entrance now."

In March 1947, at the request of the Administrative Officer, Cochin *preliminary* investigations were taken up.

A reduction in the width of the dredged channel—not impracticable from point of navigation—was, recommended. It was also suggested that from model experiments it might be possible to suggest a realignment and resectioning of the Approach Channel, which would minimize dredging, both in it as well as in the inner harbour.

The model is therefore, mainly required to answer the following questions

(1) Is the assumption correct that the main source of silting in the inner channels and particularly in the Mattancherry Channel is fine silt carried into the harbour on flood tides and deposited ?

(2) What steps can be suggested to reduce the volume of silting ? In this connection it is appreciated that the width of the Mattancherry Channel is too great to encourage a good scouring action.

(3) The model experiments are required to show whether the required depth can be maintained alongside and northwards of the wharves in the Mattancherry Channel (where it is the intention to extend the wharf facilities) without frequent dredging and what alignment and width of channel is necessary to ensure this and to give a satisfactory channel generally from the point of view of maintenance.

(4) It requires to be confirmed that the present site (south of the approach channel) where the spoil arising from the dredging is pumped and deposited is the best site to deposit the arisings and also as a dumping ground for the hopper barges from the point of view of the possibility of the material being carried into the inner harbour, Figure 2 C.33.

(5) If it is established that the silting comes from outside, would a realignment of the outer approach channel have any effect in reducing the amount of silt carried into the inner harbour ?

(6) Dredging is to be done shortly at the southern end of the Ernakulam Channel and the tankers berth proposed to be sited as shown on Figure 2 C.34. It is required to know whether these berths are sited to the best advantage and whether the tide will run true in this position.

MODEL SCALES

When selecting various scales of a model it is necessary to see that the flow in the model is fully turbulent, and there is sufficient and similar bed movement in the model. The scales of the model are interdependent and their implications on various factors will now be considered.

On considerations of the available space, suitable reproduction of all details and facility of observation in various reaches *etc.*, a length scale $\frac{1}{1,000}$ was adopted for the pilot model. From the data of *Station No. 2* in the Mattancherry Channel—where silting is known to be occurring—the values of Reynold's Number obtained with alternative proposals of vertical exaggerations, are given in Table No. 2 C.22.

$$\text{Mean depth } \frac{A}{w} = \frac{36,960}{1,488} = 24 \cdot 84 \text{ feet.}$$

Lowest maximum discharge observed on March 1940 = 14,881 cusecs.

TABLE 2C.22

V. E.	Depth Scale	Mean Depth (feet)	Mean Width (feet)	Q_m cusecs	$q_m = Vd$ cusecs foot per width	Reynold's Number
1	2	3	4	5	6	7
100	10	2·484	1·488	0·47	0·316	30,000
50	20	1·242	1·488	0·17	0·115	10,800
40	25	0·993	1·488	0·12	0·08	7,600
20	50	0·496	1·488	0·042	0·028	2,650

Note.—q_m in col. 6 is the discharge per ft. run in the model. Col. 7 (Reynold's number) = $\frac{V_d}{\nu} = \frac{q}{\nu}$ was evaluated assuming kinematic viscosity $\nu = 1 \cdot 059 \times 10^5$.

From Table 2 C.21 it is clear that a minimum V. E. of 40 is required to have a sufficiently high Reynold's Number. It was, therefore, decided to investigate further the alternative merits of V. E. of 40 and 50 in respect of their effects on other scales, bed movement *etc.*, the depth scales with these exaggerations being, in both cases, reasonably large enough to simulate the prevailing tidal range which is three feet only.

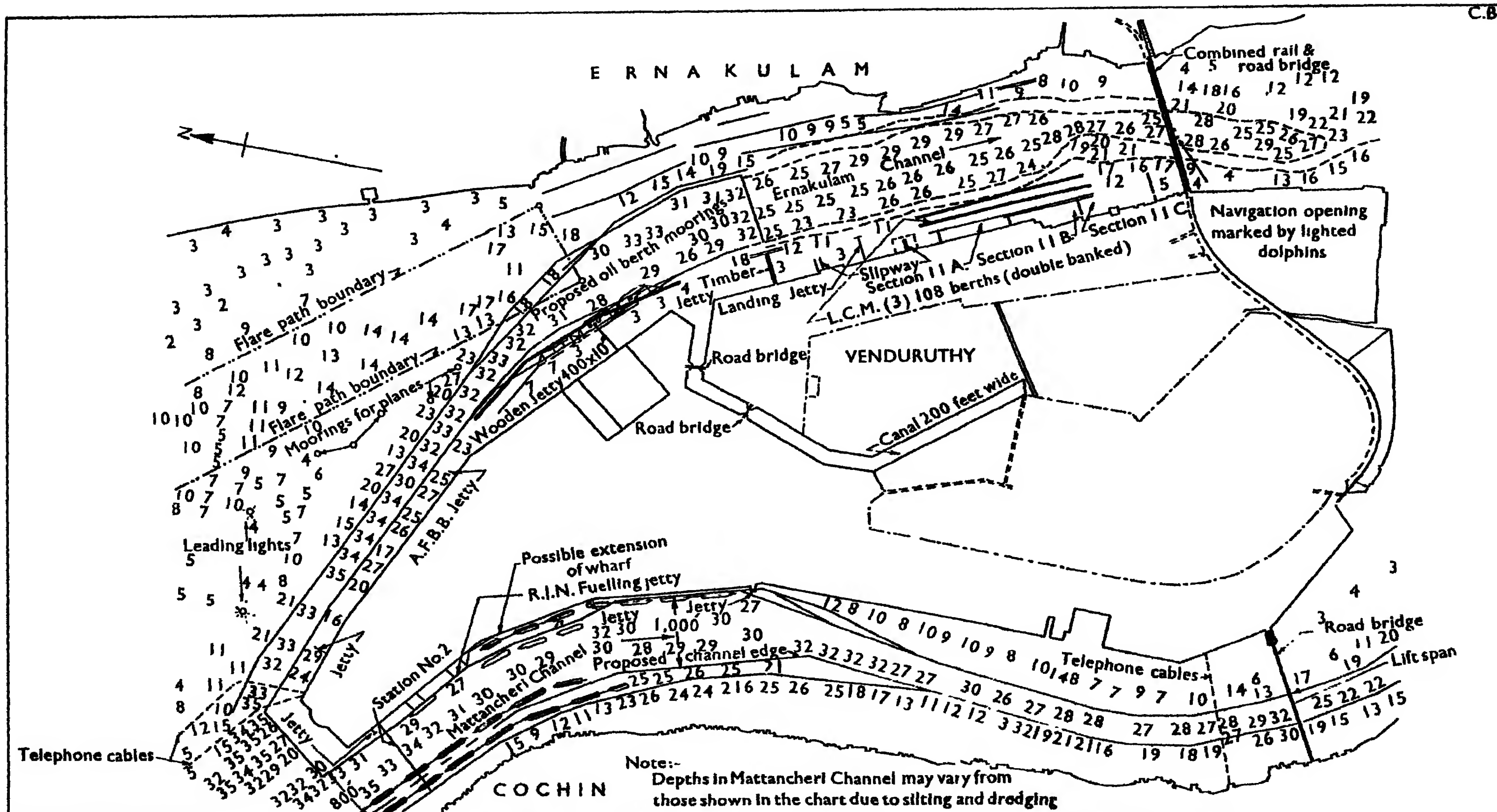

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Figure 2C.34:- *Developments in Willingdon Island and Venduruthy (Cochin Port)*

Scale 1 5 0 1 2 3 4 Thousand feet

To ensure turbulence, Reynold's criterion requires $h^3 e \sqrt{\frac{H}{30}}$ to be greater than 0·09, where h is the spring tidal range in feet at the mouth of the model ; H is the spring tidal range at the mouth of the prototype = 3 ft ; and e is the vertical exaggeration.

Thus, with V. E. = 40

$$h^3 e \sqrt{\frac{H}{30}} = \left(\frac{3}{20}\right)^3 \times 40 \sqrt{\frac{3}{30}}$$

$$= 0{\cdot}021 \text{ which is } \textit{less} \text{ than } 0{\cdot}09 ;$$

and with V. E. = 50

$$h^3 e \sqrt{\frac{H}{30}} = \left(\frac{3}{20}\right)^3 \times 50 \sqrt{\frac{3}{30}}$$

$$= 0{\cdot}052 \text{ which is also } \textit{less} \text{ than } 0{\cdot}09.$$

It will thus be seen that, even with V. E. = 50, though the Reynold's Number is much > 1,800, the value of $h^3 e \sqrt{\frac{H}{30}}$ is still less than 0·09. However, Jack Allen [39] has confirmed the Station experience that " it is highly possible to succeed with much lower values than 0·09 in the average case of the irregular and unsymmetrical estuaries found in Nature ."

Table 2 C.23 shows the values of $h^3 e \sqrt{\frac{H}{30}}$ obtained in various model investigations :—

TABLE 2 C. 23

Serial No.	Name of Model	Horizontal scale x	Vertical scale y	Vertical exaggeration e	Spring tidal range in prototype	$h^3 e \sqrt{\frac{H}{30}}$
1	(a) Reynold's own model of the Mersey ..	31,800	960	33	30′	0·001
	(b) Second model of Mersey	10,560	396	26·7		0·0125
2	Thames model by Mr. G. E. W. Grutwell ..	10,560	384	27·5	14′ 6″	0·00103
3	Seine model by Vernon Harcourt	40,000	400	100	23′ 7″	0·018
4	Severn model by Prof. A. H. Gibson	8,500 8,500	100 200	85 47·5	35′	3·8 0·27
5	Bombay Harbour by Mc. Clure	7,296	96	73	14′	0·162
6	Rangoon Model ..	8,060	192	42	16′	·017

(39) Allen J.—Scale models in Hydraulic Engineering 1947, Longmans Green & Co., page 186.

Except in the case of the Severn and the Bombay Harbour models, it will be observed, the value of $h^3\epsilon \sqrt{\frac{H}{30}}$ was *less* than 0·09 ; but the results of all these models are stated to be quite satisfactory.

It was, therefore, decided to test the effect of adopting a V. E. of 50 on other factors.

With V. E. 50, other scales work out to be as follows—

$$\text{Depth Scale} = \frac{1}{20}$$

$$\text{Discharge Scale} = \frac{1}{1{,}000 \times 20 \times \sqrt{20}}$$

$$\frac{1}{89{,}500}$$

$$\text{Time Scale} = \frac{L - \text{Scale}}{V - \text{Scale}}$$

$$= \frac{L - \text{Scale}}{\sqrt{D}\ \text{Scale}}$$

$$= \frac{\frac{1}{1{,}000}}{\sqrt{\frac{1}{20}}}$$

$$= \frac{1}{223}$$

i.e., one year in prototype = 38·8 hours in the model ; and the normal tidal period will take 3·3 minutes in the model.

DOMINANT DISCHARGE OF THE PERIYAR RIVER

The maximum discharge of the Periyar River beyond the tidal reach is 40,000 cusecs. In the year 1924 the maximum discharge ever was, however, about 80,000 cusecs. The dominant discharge may thus be assumed = 2/3 × 40,000 = 26,666 = 27,000 cusecs (approx.) not considering the abnormally high discharge. The average discharges of the Periyar through the Periyar mouth is 20,000 cusecs.

SILT CONTENTS OF THE PERIYAR

Table 2 C.24 gives the total discharge in cubic feet and the quantity of silt in cubic feet as observed at the mouth of the Periyar in 1940.

TABLE 2 C.24

Serial No.	Date of hours of observation	Total ebb discharge in cubic feet	Quantity of silt in cubic feet	Average silt charge in cubic feet per cubic foot of water
1	3 P.M. on 5-7-40 to 5 P.M. on 6-7-40	$4{,}380 \times 10^6$	874,800	0·000199
2	11 A.M. on 22-7-40 to 1 P.M. on 23-7-40	$3{,}824 \times 10^6$	135,000	0·00035
3	12 noon on 2-8-40 to 1 P.M. on 3-8-40	$4{,}995 \times 10^6$	523,800	0·000105
4	11-30 A.M. on 23-8-40 to 1-30 P.M. on 24-8-40	$4{,}239 \times 10^6$	315,900	0·00074
5	11-30 P.M. on 6-9-40 to 12-30 P.M. on 7-9-40	$1{,}359 \times 10^6$	24,300	0·000179
6	11-30 A.M. on 4-10-40 to 1-30 P.M. on 5-10-40	$2{,}004 \times 10^6$	410,400	0·000206 *Mean*=0·000106

The weight of one cubic foot of Periyar silt=80 lb. The silt contents of the Periyar will, therefore, be

$$\frac{80 \times 0\cdot000106}{62\cdot4} \times 1{,}000 = 0\cdot13 \text{ part per 1,000 by weight.}$$

Assuming the same silt charge in the model, the quantity of silt required for 6 hours working-day in the model

$$= 0\cdot224 \times 6 \times 3{,}600 \times 62\cdot4 \times 0\cdot000106$$

$$= 32 \text{ lb.} = 0\cdot4 \text{ cubic foot.}$$

As an indication whether the motion of water in the model will be turbulent, J. Allen (40) suggests the criterion *viz.*,

y should not be *greater* than $20\,(v\,H)^{\frac{2}{3}}$, where v is the maximum current velocity in knots during flood tide (usually occurring at about half-tide)

(40) Allen J : 'Scale Models in Hydraulic Engineering,' 1947, page 279.

H the *mean* depth in the channel and y is the vertical scale. This offers, according to him, a reasonably safe check upon the chosen vertical scale from the aspect of turbulence, especially if v and H relate to neap tides.

At the mouth of the estuary at Cross-Section No. 1 the following observations are available regarding water level and discharges during Non-monsoon and Monsoon Neap tides.

Non-monsoon neap tide on March 15, 1940.

On half flood tide, water level = 2·1 feet above low water ordinary spring tide.

Discharge = 10,000 cusecs.

Area of cross-section No. 1 at L. W. O. S. T. on March 15, 1940 is not available but on April 22, 1940 it was 51,980 sq. ft.

Assuming practically the same area as on March 15, 1940 the area of section up to 2·1 feet above L. W. O. S. T. will be

$$51{,}980 + 2{\cdot}1 \times 1{,}300$$

$$= 54{,}710 \text{ square feet.}$$

$$\therefore \text{Velocity} = \frac{10{,}000}{54{,}710} = 0{\cdot}18 \text{ feet per second} = 0{\cdot}11 \text{ knot.}$$

$$H = 38{\cdot}5 + 2{\cdot}1 = 40{\cdot}6 \text{ feet.}$$

$$\therefore 20\,(v\,H)^{2/3} = 20\,(0.11 \times 40.6)^{2/3}$$

$$= 20 \times 1{\cdot}95 = 39$$

$\therefore y = 20$ is *not greater* than $20\,(v\,H)^{\frac{2}{3}}$ and thus satisfies the criterion.

Similarly, for monsoon neap tides on September 25, 1940 on half flood tide

Water Level = 1·6 feet above L. W. O. S. T.

Discharge = 30,000 cusecs.

Area of cross-section No. 1 on November 27, 1940 was 54,210 sq. ft. Assuming this area on September 25, 1940 the area of cross-section up to water level, 1·6 feet above L. W. O. S. T, will be

$$54{,}210 + 1{\cdot}6 \times 1{,}300 = 56{,}290 \text{ sq. feet.}$$

$$\therefore v = \frac{30{,}000}{56{,}290} = 0{\cdot}53 \text{ ft./sec.} = 0{\cdot}31 \text{ knot.}$$

$$\therefore 20\,(v\,H)^{\frac{2}{3}} = 20\,(0{\cdot}31 \times 40{\cdot}1)^{\frac{2}{3}} = 108.$$

$\therefore y = 20$ is *not greater* than $20\,(v\,H)^{\frac{2}{3}}$

An analysis of experiments made in straight-sided channels of rectangular cross-section indicated that the mean velocity of the stream $\overline{V}$ ft./sec. required to move grains forming an initially flat bed is given by

$$\overline{V} = K\left(\frac{L+B}{21}\right)^{0.44} (\sigma' - \sigma)^{0.22} \quad l^{0.22} \quad M^{-0.22} \quad h^{0.06} \quad m^{0.22}$$

where $K = 1{\cdot}59$ for movement of many particles.

$\frac{L+B}{2}$ = mean dimension of the grains as viewed under a microscope (inches).

l = length of side of a cube having the same weight as the average grains (inches);

σ' = specific gravity of material;

σ = specific gravity of water (say unity);

M = uniformity modulus of material;

h = depth of water (inches);

and m = hydraulic mean depth of stream (inches).

The following are the observations of L and B under a microscope of sea sand sieved through 20 mesh and Londha sand sieved through 30 mesh. These sands were considered to be suitable for these experiments and were, therefore, analysed in detail as shown in Table 2 C.25.

TABLE 2C-25.

No. of particles	Length of particle		Breadth of particle	
	Sea-sand mm.	Londha sand mm.	Sea-sand mm.	Londha sand mm.
1	0·04	0·024	0·03	0·023
2	0·09	0·046	0·045	0·045
3	0·02	0·043	0·017	0·030
4	0·024	0·030	0·013	0·030
5	0·042	0·050	0·018	0·040
6	0·055	0·055	0·018	0·035
7	0·031	0·035	0·025	0·024
8	0·020	0·040	0·012	0·023
9	0·025	0·056	0·017	0·045
10	0·065	0·051	0·040	0·039
11		0·019		0·015
12		0·013		0·012
Mean	= 0·041	0·038	0·023	0·030

$\therefore \quad \dfrac{L+B}{2} = 0\cdot032$ mm.

$= 0\cdot0012$ inch for sea sand ; and

$\dfrac{L+B}{2} = 0\cdot034$ mm.

$= 0\cdot00133$ inch for Londha sand.

If $\frac{L + B}{2}$ is less than 0·015 inch, as this case, then

l may be computed sufficiently accurately from the formula ;

$$l = \frac{(L + B)}{2} \quad \frac{B}{L}$$

$$= 0{\cdot}015 \times \frac{0{\cdot}023}{0{\cdot}041}$$

$$= 0{\cdot}00067 \text{ inch for sea sand ; and}$$

$$l = 0{\cdot}00133 \times \frac{0{\cdot}03}{0{\cdot}038}$$

$$= 0{\cdot}00089 \text{ inch for Londha sand.}$$

For sea sand $\sigma' = 2{\cdot}62$ and

for Londha sand $\sigma' = 2{\cdot}64$

M for both sands $= 0{\cdot}75$

The depth of water in prototype

= 40 feet.

= 2 feet in model.

$$m_{\text{proto}} = \frac{A}{P} = \frac{A}{W} = \frac{52000}{1300} = 40$$

m_{model} = 5·0 inches (found out by plotting the section).

$$\therefore \quad \bar{V}_{\text{sea sand}} = 1{\cdot}59 \left(\frac{0{\cdot}0012}{0{\cdot}00067} \right)^{0{\cdot}44} \times (2{\cdot}62 - 1)^{0{\cdot}22} \times (0{\cdot}00067)^{0{\cdot}22}$$

$$\times (0{\cdot}75)^{-0{\cdot}22} \times (24{\cdot}0)^{0{\cdot}06} \times (5{\cdot}0)^{0{\cdot}22}$$

$$=\frac{1\cdot59\times1\cdot29\times1\cdot11\times0\cdot2\times1\cdot21\times1\cdot425}{0\cdot88}$$

$= 0\cdot9$ foot per second.

$$\bar{V}_t \text{Londha sand} = 1\cdot59\left(\frac{0\cdot00133}{0\cdot00089}\right)^{0\cdot44}\times(2\cdot64-1)^{0\cdot22}\times(0\cdot00089)^{0\cdot22}$$

$$\times\,(0\cdot75)^{-0\cdot22}\times(24\cdot0)^{0\cdot06}\times(5\cdot0)^{0\cdot22}$$

$$=\frac{1\cdot59\times1\cdot19\times1\cdot11\times0\cdot212\times1\cdot21\times1\cdot425}{0\cdot88}$$

$= 0\cdot87$ foot per second.

Considering, now, the Mound formula (2), (given on p. 280 of Allen's Book) for the first disturbance of the particles forming a mound of material, namely

$$v = 1\cdot30\,(\sigma'-\sigma)^{0\cdot27}\,l^{0\cdot27}\,m^{-0\cdot27}\,h^{0\cdot23}$$

$$\bar{v}\text{ (sea sand)} = \frac{1\cdot3\,(1\cdot62)^{0\cdot27}\,(0\cdot00067)^{0\cdot27}\,(24\cdot0)^{0\cdot23}}{(0\cdot75)^{0\cdot27}}$$

$$=\frac{1\cdot3\times1\cdot139\times0\cdot14\times2\cdot07}{0\cdot88}$$

$= 0\cdot49$ foot per second.

$$\bar{v}\text{ (Londha sand)} = \frac{1\cdot3(1\cdot64)^{0\cdot27}\,(0\cdot00089)^{0\cdot27}\,(24\cdot0)^{0\cdot23}}{(0\cdot75)^{0\cdot27}}$$

$$=\frac{1\cdot3\times1\cdot143\times0\cdot15}{0\cdot88}\times2\cdot07$$

$= 0\cdot525$ foot per second.

Maximum flood velocity in model

$$=\frac{2\cdot53\times1\cdot69}{4\cdot49}=0\cdot95 \text{ foot per second.}$$

Maximum ebb velocity in model

$$= \frac{2\cdot93\times1\cdot69}{4\cdot49} = 1\cdot1 \text{ foot per second.}$$

Table 2 C.26 gives the velocity to move many particles, velocity for the first disturbance of particles forming a 'mound,' and the model velocities corresponding to the maximum ebb and flood velocities in the prototype :—

TABLE 2 C.26

Type of sand	$\bar{v}$ for many particles to move feet per second.	$\bar{v}$ according to Mound formula feet per second.	Model Velocities	
			Maximum flood feet per second.	Maximum ebb feet per second.
Sea-Sand (Sieved through 30 mesh); $m = 0\cdot191$ mm.	0·9	0·49	0·95	1·1
Londha Sand sieved through 30 mesh; mean dia. = 2·04 mm.	0·87	0·525	0·95	1·1

Table 2 C.26, therefore, shows that the velocities in the model will be sufficient to move the bed particles and the vertical scale of 1/20 will be satisfactory from the aspect of bed movement. Both the sands analysed are suitable in this respect but sea-sand has been preferred being finer. For the mud banks and the mud bed, the *original* mud banks material from Cochin is pro posed to be used.

Adopting the V. E. = 50 and the depth scale = $\frac{1}{20}$, the silt in the model should have a vertical motion $\frac{1,000}{(20)^{\frac{3}{2}}} = 11\cdot1$ times as fast as in the prototype.

Table 2 C.27 shows this factor in other models ;—

TABLE 2 C.27

Name of the model	Horizontal Scale x	Vertical Scale y	$\frac{x}{y^{\frac{3}{2}}}$
1. Mersey (Manchester—University) ..	7,040	190	2·68
2. Rangoon River	8,060	192	3·04
3. River Parrett	3,000	260	0·716
4. Severn	8,500	200	3·0
5. Dee	(a) 5,000	25	40·0
	(b) 40,000	100	40·0
6. Great Ouse & Wash..	2,500	41·7	8·95

In comparison with the first four models, it will be seen that a faster rate of fall of water-borne silt is required in this model. The V. E., however

cannot be reduced to get the rate of fall equal to three times that in the prototype as the important scales like depth scale, discharge scale would then become unsuitable. Alum will, of course, be essential and arrangements are made for re-circulation to economise in cost of alum:

The following scales are, therefore, tentatively, decided to be adopted for the pilot model:

Length Scale	$= \frac{1}{1,000}$
Depth Scale	$= \frac{1}{20}$
V. E.	$= 50$
Discharge Scale	$= \frac{1}{89,100}$
Time Scale	$= \frac{1}{223}$
Velocity Scale	$= \frac{1}{4{\cdot}49}$

Adopting these scales the tidal curves obtained for the model, corresponding to the tidal curves observed in the harbour in 1940, will be as shown in Figure 2 C.35. These tidal curves are for Stn. No. 1 at the Gut. The curves have been plotted assuming the speed of the recorder as one inch of the periphery in four minutes.

The tide data is being yet analysed to study the representative tides which must be reproduced to simulate in magnitude, order and frequency the integrated tidal effect.

The Survey Research Institute, Survey of India, furnished a note on the main features of the tides at Cochin.

OTHER RELEVANT FEATURES

The coastal currents.—The coastal currents are supposed to run south for 8 months in the year, to be more or less stationary for a month and to run north for the remainder. The Meteorological Atlas shows varying speeds, roughly in proportion with wind velocity. There is practically complete agreement among sailors that the north to south drift has been much stronger since 1934 or 1935. Notwithstanding this, the sand and shells continue to move north with the ground swell.

The opposing force is the south-west swell set up by wind waves before and at the beginning of the S. W. Monsoon.

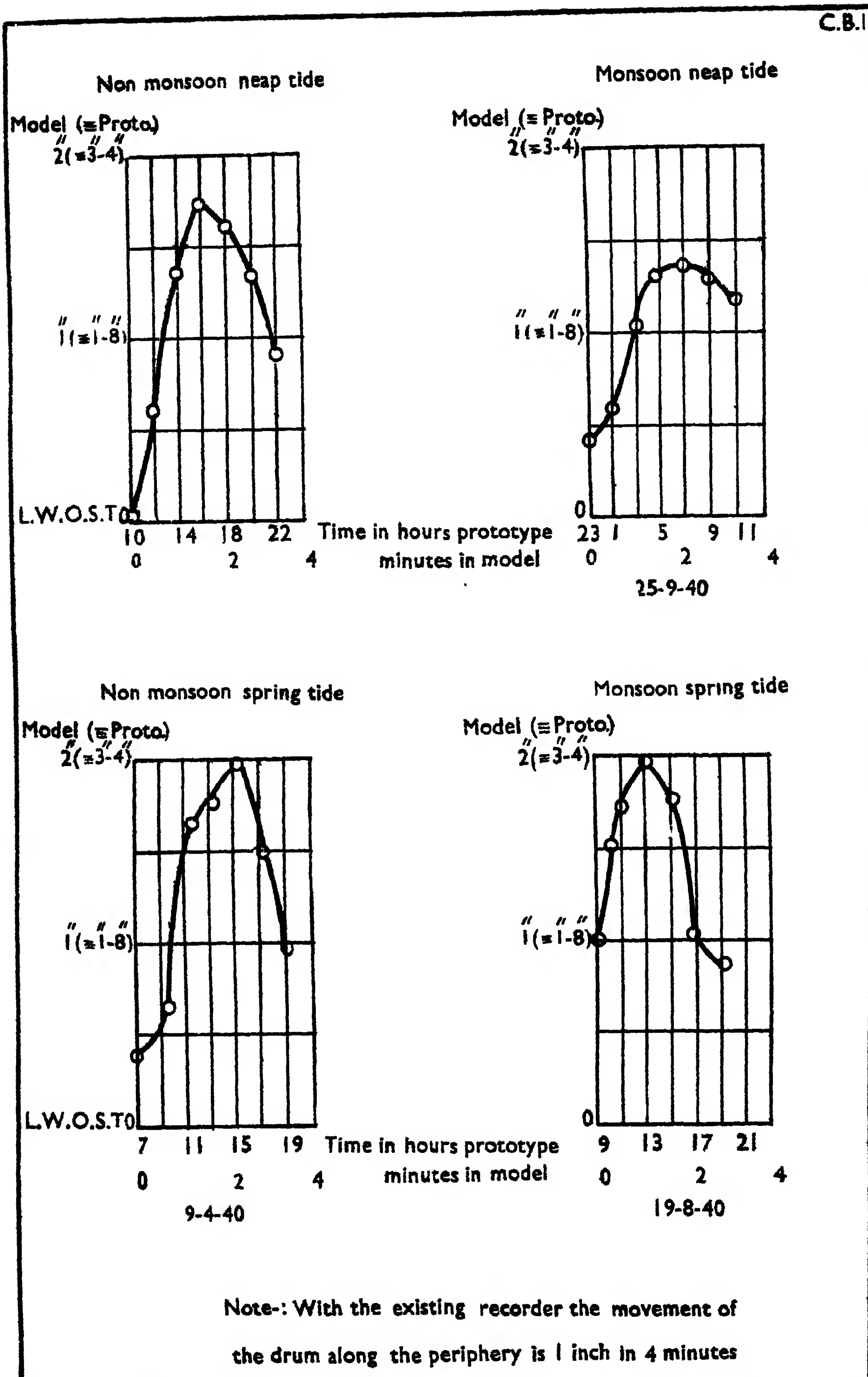

Figure 2C.35:- *Showing tidal curves at station No.1 Cochin Harbour Model*

Reg. No. 2737 XDD (C)' 50 - 1,210 (P.Z.O.).

Soundings off Cochin.—(*a*) from 1835 to 1913 the contours may be reckoned either as fairly stable or returning to about their normal position if temporarily displaced between the dates of charts ;

(*b*) at some time unknown between 1913 and 1920 there was a shallowing but of no material importance ;

(*c*) between 1920 and 1931, the shallowing became obvious specially to the north of the harbour entrance ;

(*d*) after 1933 or 1934 the shallowing to the north increased, and rapidly so in 1936-38 ; and

(*e*) on the contrary, 1½ miles south of the Flag-staff, the contours returned to their original line of 100 years ago, or near it.

It is of interest that the rate of shallowing coincide with the approach of the Narakal bank towards the harbour entrance ; the nearer the bank the quicker the shallowing.

Soundings in Outer Channel.—The depths in the channel before and after the monsoon are given in Table 2 C.28 The most marked difference occurs in 1937 when the average sounding for the whole channel was the lowest on record except in 1928.

TABLE 2 C.28

Year	Measured depths in outer channel			
	After dredging average feet L. W. O. S. T.	After monsoon average feet L. W. O. S. T.	Least depth feet L. W. O. S. T.	Silting in millions cubic yards
1928	34·0	27	24	1.26
1929	35·9	32·7	29.3	0.87
1930	35·9	32·1	28.8	1.06
1931	36·5	34·1	31.6	0.65
1932	36.5	34.1	31.0	0.65
1933	39.8	33.5	30.5	1.54
1934	39.6	31.6	29.5	2.02
1935	38.4	31.3	29.3	1.89
1936	38.5	31.4	29.8	1.95
1937	38.5	29.9	28.8	2.37
1938	38.2	33.1	30.8	1.7
1939	38.25	33.4	30.2	1.65
1940	38.2	33.1	31.5	1.5

The data are still under further investigation. The contruction of the model is started. It is fully realised that this model investigation will be very greatly complicated by the unique phenomena of the formation, movement and the calming effect of the mud banks. The difficulties of simulation of these affects in the model will be great but they can be surmounted, it is believed, by suitable model technique and by making the allowances for model limitation.

DISCUSSION BY THE RESEARCH COMMITTEE

RAO BAHADUR D. V. JOGLEKAR introducing item (1) said that in the case of the knuckle at Titaghar Jute Mill, the turbulence downstream was due to the " change over " of the flow occurring immediately upstream of the knuckle. In addition to this, there was throttling of the width of the river channel immediately downstream of the knuckle. The imposition of these rather severe conditions made the action of a ' row of piles '—found successful in the case of the Dunbar Cotton Mill knuckle– ineffective at Titaghar in reducing the turbulence and inducing silting in the bight.

Experiments were carried out on a $\frac{1}{300}$: $\frac{1}{60}$ scale model, in which the lines of flow and velocity distribution were obtained, and similar conditions were then reproduced in $\frac{1}{150}$: $\frac{1}{80}$ scale part-length and less exaggerated model in which silting and scouring effects were observed with injection of fine silt during ebb tide—conditions closely analogous to prototype conditions.

It was found that a single island suitably located was more effective in attracting the flow on the right bank shoal. Islands placed upstream or downstream of this island were found to be either more or less ineffective or doing more harm than good due to the changing conditions of the tide.

Basic experiment to see the effect of the bottle-neck downstream of the knuckle were then made. Widening the river at the existing bottle-neck, was not a practical proposition due to vested interests ; but the experiment was done to verify the assumption that the severe conditions were partly due to this ' bottle-neck '. These experiments showed that widening the river at the bottle-neck considerably reduced the turbulence at the knuckle and the knuckle would not have formed but for these conditions.

MR. S. T. GHOTANKAR then introduced item (2) and said that in the years 1927-29, a sea approach channel to the Cochin Port, 450 feet wide and 16,000 feet long was dredged to a depth of 37 feet below low water of ordinary spring tide. In the inner harbour, the Mattancherry Channel, mooring areas and the Ernakulam Channel were dredged. Since 1933, there had been greater silting in the approach channel and in the Mattancherry Channel. It was first thought that the probable cause of this silting might be the low rainfall since 1933, which had reduced the scouring effects of the monsoon freshets. In 1937, however, despite twice the normal rainfall, there was inordinate silting and it became necessary to investigate further the cause of this heavy silting.

After diagnosing the cause of silting of the Mattancherry Channel it was recommended to reduce the dredged width of the Mattancherry Channel—practicable from the point of navigation—with a view to reduce the volume of sedimentation in this channel due to its excessive section, pending further recommendations to result from model studies.

DR. N. K. BOSE said that it was stated " In the river there is generally about 10 per cent. of bed load and 90 per cent. of suspended load ". He thought this was incorrect and this was exactly opposite of what he thought was actually the case. Had any data been collected to show this ?

RAO BAHADUR D. V. JOGLEKAR replied that this had been worked out from cross-sections on the Rhine River and someone from China had quoted a similar case. He was quite sure that the bed load was of the order of 10 per cent. though he could not quote exact reference off hand.

DR. N. K. BOSE then said that he would like to say a few words on the subject. The excellent note put up explained the problem well. The model, however, would be an extremely complicated one. The following points would have to be reproduced in the model :—

(1) Tidal Flow and Waves
(2) Ground swell
(3) Littoral drift
(4) Sea wind
(5) Cyclone
(6) Fresh water flow
(7) Sand movement
(8) Supply of fine clay and its interaction with the saline water of the sea
(9) Mud bank, its physical and chemical characteristics.

It was proposed to use the original material from the mud banks. Care should be taken to see that the chemical character of this material did not change in transit to the Research station.

Considering the complexity of the problem and the various factors : meteorological, chemical and physical interacting, it would have been better if the model had been operated at site. DR. BOSE as a word of caution said that extreme care would have to be taken to see that the mud brought from the 'Mud Bank' at site did not change its chemical character during transit which it was likely to do.

DR. N. K. BOSE asked whether the cross flow across the nose of the left divide wall would not make the approach to the lock as proposed very unsafe at least for country boats ? Extreme care would have to be taken to negotiate this approach.

DISCUSSION BY THE BOARD

THE SECRETARY said that two items were discussed at the Research Committee meeting (page 701). There was no resolution.

It was decided to retain these items on the Agenda.

3C. Reservoirs

PRELIMINARY NOTE

At the 1946 Research Committee Meeting, it was recommended to the Board that the subject ' Silting of reservoirs ' be changed into "Reservoirs" with two sub-heads, (*i*) Silting of reservoirs (*ii*) Losses from reservoirs.

In 1947 it was decided that the sub-head ' Losses from reservoirs ' should go under Section A—Hydrology, item 5, ' Regeneration and losses ' and the following two sub-heads were adopted for the subject head " Reservoirs ".

(*i*) Sedimentation of reservoirs

(*ii*) Capacity Surveys

Sedimentation of reservoirs

PR IMINARY NOTE

This subject has been on the agenda since 1939.

A comprehensive resolution* on this subject was passed by the Board at its 1946 meeting. This resolution was forwarded to administrations. The replies from Sind, Cochin, the United Provinces and Bombay show that instructions have been issued to the Officers concerned for implementing the recommendations of the Board. The Central Provinces, Madras and Ceylon replied that due to shortage of staff it was not possible for them to take up the work of observations for the present. The Chief Engineer Jodhpur has raised the following points :—

(1) Due to shortage of staff Jodhpur cannot take up the observations in accordance with the recommendations of the Board, but if information pertaining to measured depths of silt deposit obtained by digging the beds of tanks which are dry, the ages of which are known accurately, are going to be of value, the Chief Engineer would be prepared to obtain these.

(2) Regarding the advancement of delta into tank beds, he has suggested that the only satisfactory way of recording would be by means of air photos taken at five years' intervals over a representative series of tanks ; this in his opinion should not be either difficult or costly to arrange.

*The Central Board of Irrigation Journal, April 1947, page 136.

(3) Since there is very little silt in Rajputana Rivers, yearly measurements of silt by levelling or soundings will not show significant changes. He has, therefore, suggested that the interval should be five years.

(4) For flashy rivers silt sampling is difficult to arrange and the information to be collected will not commensurate with the cost and trouble involved.

The Board examined the points raised by the Chief Engineer, Jodhpur at its 1947 meeting and after discussion decided as follows :

(1) The information so obtained would be very useful and Chief Engineer, Jodhpur, be requested to collect it. He should also be requested to ascertain, if possible, the dry bulk density of silt, *in situ* by layers in its wet (spongy state) and whether it is possible to distinguish the various layers yearly or otherwise.

(2) A good suggestion which might be adopted for all reservoirs. The period between two successive photographs would vary for each case.

(3) The period would vary for different places and must be determined by local officers.

(4) Agreed.

The publication of " Silting of Reservoirs " by Rai Bahadur A. N. Khosla as a Board Publication was also approved.

The following resolution was passed by the Board at its 1947 meeting :—
" Resolved that all Provinces and States be requested that :

(*i*) As an ultimate objective, but for the present, as far as possible at discharge observation sites on main and tributary streams, sediment discharge observations, with analysis into principal sizes, should be made concurrent with every water discharge observation. The results should be correlated annually with erosion intensity surveys done by the Forest Department.

(*ii*) Every project for damming a river system should contain a report detailing, in quantitative terms, the whole relevent sediment data and the economics of action required to protect the reservoirs from rapid loss of capacity, and the rivers from undesirable regime changes.

The Board further recommends that soil conservation experts be appointed from the investigation stage to each reservoir project. "

In response to this, Jodhpur has promised to give effect to the resolution and to supply some data from the Aravally Hills which might give some useful information.

The following items were discussed at the 1947 Research Committee meeting :

(1) Observation of suspended load at Dhukwan.

(2) Relation between rate of silting and rainfall.

(3) Relations between rate of silting and catchment area.

(4) A note on " Silting of Reservoirs "—by Mr. B. P. Saxena.

Recent Literature.

(1) Gottschalk L. C.—Silting of stock ponds in land utilisation project area SD-LU-2 Pierre, South Dakota—U.S. Department of Agriculture, Soil Conservation Service, Special Report, No. 9, Washington, D. C., May 1946.

(2) Silting of Reservoirs—Central Board of Irrigation Journal, Vol. 4, No. 2, April 1947.

(3) Harty V. D.—Silting investigations for reservoirs and dams—The Surveyor, Vol. 105, No. 2821, February 15, 1946.

(4) Raju S. P.—Nizamsagar silt problem—H.E.H. The Nizam's P.W.D., Hyderabad Deccan.

(5) Malhotra J. K.—Formula for the rate of silting of the Elephant Butte Reservoir—Central Board of Irrigation Journal, Vol. 5, No. 1, January 1948.

(6) Vetter C. P., Chief Office of River control, Region 3, Bureau of Reclamation, Boulder City, Nevada—Silt Problems on the Colorado River—International Commission on Large Dams, Third Congress, Stockholm, 1948, C7.

THE YEAR'S WORK

The following item was discussed at the 1948 meeting of the Research Committee.

(1) Rate of silting of reservoirs.

(1) RATE OF SILTING OF RESERVOIRS ([1])

One of the major questions facing the designer of a reservoir is: "At what rate will it silt up?" As a rider he might add: "Will the rate of silting decline with the passage of years?"

For if it does, the effective life of the reservoir might be much longer than if the rate of silting remained constant at its initial value.

While the rate of silting for a projected reservoir can be tentatively fixed in the light of local knowledge, the available data for existing reservoirs might be usefully studied to get some indication of similar behaviour elsewhere.

The only reliable long term data for the rate of silting in a fairly large reservoir appear to be given by Stevens ([2]). These relate to the Elephant Butte Reservoir on the Rio Grande, which has a capacity of 2·6 million acre feet.

The results of the surveys of sediments deposited in this reservoir below spillway level, as summarized by Stevans, are given in Table 3 C.1.

TABLE 3 C.1

Period	Years	Cumulative years	Deposit (thousands of acre feet)		Capacity lost (%age of original
			Period	Cumulative	
Storage began (January 6,1915) ..	0	0	0	0	0
Jan. 1915—Dec. 1916	1·91	1·91	54·0	54·0	2·04
Dec..1916—Aug. 1920	3·67	5·58	86·0	140·0	5·40
Aug. 1920—Aug. 1925	5·00	10·58	91·7	231·7	8·78
Aug. 1925—Apr. 1935	9·67	20·25	133·5	365·2	13·8
Apr. 1935—Sept. 1940	3·50	25·75	50·6	415·8	15·8

Stevens plotted 'the capacity lost' against 'cumulative years' and noted that 'the rate of deposition in the reservoir appars to have been decreasing materially'. Somewhat curiously, however, he concluded that there was 'insufficient evidence to warrant the adoption of other than an average linear rate of filling.'

As the above did not seem to be quite consistent, the data were re-analysed n the Institute.

([1]) East Punjab Irrigation Research Institute, Amritsar, Annual Report, 1947, pages 21-23.

([2]) Future of lake Mead and Elephant Butte Reservoirs, Proceedings of the American Society of Civil Engineers, Volume 71, No. 5.

The 'cumulative deposit' was plotted against 'cumulative years'. All the six observed points seemed to lie on a smooth curve, and the adoption of the linear rate of filling did not, therefore, seem justified.

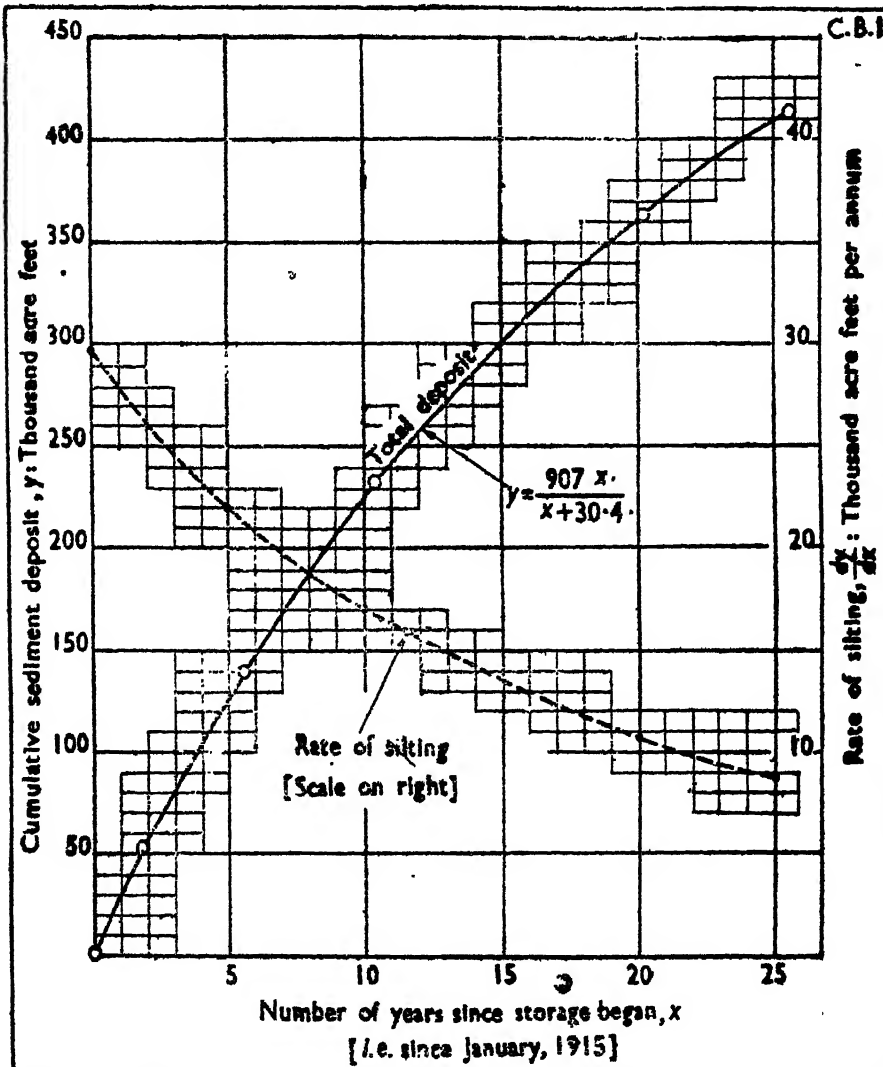

Figure 3 C.1

The equation of this curve, Figure 3 C.1, was found to be given by the formula.

$$Y = \frac{907 \cdot x}{x + 30 \cdot 4} \quad \ldots \quad \text{(3C.1)}$$

where y = cumulative sediment deposit, in thousands of acre feet, and x= number of years since January 1915.

The values given by Equation (3C. 1) compared as given in Table 3 C. 2 with the actuals :—

TABLE 3C. 2

Cumulative years	Cumulative deposit (thousand acre feet) Observed	By formula (3·1)
0	0	0
1·91	54·0	53·6
5·58	140·0	140·7
10·58	231·7	234·2
20·25	365·2	362·6
25·75	415·8	415·9

Considering that the observations covered a period of 25 years, the fit would seem to be very close.

The rate of silting was obtained by differentiating Equation (3C. 1) as

$$\frac{dy}{dx} = \frac{27,570}{(x+30 \cdot 4)^2} \quad \text{(3C. 2)}$$

Obviously it has been decreasing very appreciably with time, a period of 25 years serving to reduce it from the intial 30·0 to 9·0 thousand acre feet per annum.

While it need not be concluded from this one example that the rate of silting of a reservoir must inevitably decline, as time passes the necessity for periodical capacity surveys on all large reservoirs both existing and projected and careful analysis of the resulting data should be fairly obvious.

It would be of interest to learn from the engineers in charge of Elephant Butte Reservoir, whether any survey has been carried out after 1940, and if the results thereof satisfy Equation (3C. 1).

Up to January 1948, the cumulative deposit according to this formula should be 472·1 thousand acre feet.

DISCUSSION BY THE RESEARCH COMMITTEE

Dr. J. K. Malhotra introduced the item and said that as the note had already appeared in the Central Board of Irrigation Journal he would in his introductory remarks touch only one or two salient points. The study was undertaken before the partition of the Punjab in order to get some idea whether the rate of silting of reservoirs changed with time. No data were available for Indian reservoirs. Therefore he examined American data and the only available data for a long period for a fairly big reservoir were those for the "Elephant Butte Reservoir" figures which were available right from the beginning in 1915 upto 1940, *i.e.*, for a period of 25 years. Mr. J. C. Stevens, President of the American Society of Civil Engineers wrote an article on this in the Journal of the Society for May 1915 and therein stated that a linear rate of silting appeared to be borne out by the data. It appeared to the speaker that the rate of silting declined with time and so he had worked out a formula which fitted the data very closely and showed that in these 25 years the rate of silting had gone down from 30 thousand acre feet to 9 thousand acre feet per annum. From this one case, one should not, however, conclude that in any other case also the rate of silting would decline with time. That depended on the local factors which existed and a very careful study of each individual case was necessary. He thought that they would have to ask the Americans whether the formula evolved by him applied to the Elephant Butte data subsequent to 1940. In the future reservoirs and any of those at present existing, capacity surveys would have to be duly carried out and recorded so that they had some information whether the rate of silting declined with time. This was a very important point because if the silting declined with time, it affected the financial aspects of the reservoir and scheme to a very great extent.

Rai Bahadur S. D. Khangar said that capacity surveys had been carried out in respect of a number of reservoirs, from time to time in India also, and enquired whether these surveys also showed this decline in rate of silting with time. He also enquired as to how the measurements of suspended silt correlated with actual silting of the reservoir. So far they had devised no means of measuring bed silts. They had information only on the suspended silt. Could observations on the rate of suspended silt be correlated with actual rate of silting the reservoir would be very useful ?

Mr. S. L. Malhotra said that in the Punjab, they did some analysis for the Otu Reservoir in Hissar district. It was a very desirable thing to measure and find out how much silting had taken place and what were the practical difficulties in the process ; whether it was done by actual surveying or contour surveys *etc*. The analysis of data for Otu Reservoir was actually being done by Dr. Malhotra who would say something on the subject.

Mr. T. S. Venkatarama Iyer enquired as to how the measurement of silting was actually done in case of Elephant Butte Reservoir—by contour survey or by some other method.

Rai Bahadur M. C. Bijawat said that they had a reservoir in Bundhelkhand, constructed in 1910-11, and he was incharge of that for about four or five years. He was asked to carry out the survey during the rains to measure the amount of silt in suspension. Later in March or April the reservoir was dewatered and an actual survey of the silt deposit was made. With these two results the life of the reservoir was calculated for nearly 50 to 60 years at that time. Since then, 30 years had passed. He did not think the reservoir had lost even one-third of original capacity. Hence with the passage of time, the rate of silting in the reservoir had decreased.

After all the slopes in the bed of the river must change after the construction of the reservoir and that should stop first of all the silt yet to be collected in the reservoir, and the amount of silt carried from the catchment should also decrease with time and change of configuration. It required a quantitative analysis, and he presumed enough data would be available for the purpose in India. Most probably some such surveys had been done in Madras whose data should be analysed.

Mr. B. P. Saxena said that several surveys had been done in United Provinces and the results recorded. He had mentioned the same in the Research Committee meeting in the previous year. The formula could only apply to a particular reservoir. Of course, it depended on the percentage of runoff and storage, and so many other factors. It might be very useful to try this formula on other reservoirs.

The Secretary said that he wanted to make one or two remarks regarding methods of doing capacity surveys of reservoirs. On this subject, last year, instructions were issued by the Board to provinces. They proposed to issue a publication from the Board on the silting of reservoirs. They were going to get all the data that had been collected so far from the various Provinces ; mostly U.P., C.P., Bombay and some states. Unfortunately there was no correlation between the suspended silt and bed silt. In any case the methods of determining the suspended silt were in a state of development. They were going to issue certain standards now for their own use.

Dr. R. C. Hoon said that he thought it was a very valuable conclusion that Dr. Malhotra had drawn. It seemed that the density currents which were formed in such reservoirs might be one of the causes of decreased rate of silting with time in those reservoirs.

Sardar Bahadur Sarup Singh said that he thought the most important factor was the change in the density of the silt. The density of the lower levels went up and that was the chief cause for the decrease of the rate of silting in the reservoirs as years went by.

The Chairman (Rao Bahadur A. R. Venkatachari) said that the subject was a very important one, and they could not yet say anything one way or the other. It might be that this particular reservoir showed a gradual reduction in the rate of silting and it was very probable it might be so generally in all reservoirs. It might possibly also be that after a certain number of years

detailed study of Dr. Hoon's suggestion they might learn that the density current were directly responsible for this. They should collect more data and analyse whatever data were available before they could draw any useful conclusion. Even in that one case, it was rather difficult to follow the other features of the catchment *etc.* He did not recollect at the moment whether it was a regular earth dam with a high level escape or it was a masonry dam. All these factors would have considerably relevant to the results which had been analysed. The original proceedings of the Institution of Civil Engineers would give all that information and unfortunately no action had been taken in that line at any great length.

Dr. J. K. Malhotra referring to Rai Bahadur Khangar's query regarding suspended silt and bed silt correlation said that there were not enough data available to establish the correlation. It was much better to go through the data which were available and then see whether there was decline or not. Once they had established that that was applicable to a particular reservoir or generally, they could go ahead. One of the reasons why they had taken the data for that particular reservoir was its fairly big reservoir capacity, comparable to the Bhakra. Similar reservoirs might or might not behave in the same way. As regards the Otu Reservoir in the Yamuna Canal Circle, he had noted from his tentative analysis that there had been a tentative decline in the rate of silting.

With reference to the point raised by Mr. B. P. Saxena he said that the particular formula applied to that particular reservoir. He did not suppose that it would apply to all reservoirs. It might be useful, however, to apply it to other case and see if it or something similar gave a reasonable fit to the data.

Rai Bahadur S. D. Khangar suggested to carry out investigations of suspended silt load of existing reservoirs and correlate with actual rate of silting of reservoir.

The Secretary replied that the suggestion had been already incorporated in the Board's resolution.

Sardar Bahadur Sarup Singh enquired if the principle enunciated by the formula would apply to other cases.

Dr. J. K. Malhotra replied that it was difficult to say that. It was for other workers to examine the existing reservoir data and say if the silting declined with time.

DISCUSSION BY THE BOARD

THE SECRETARY said that only one item was discussed at the Research Committee. There was no resolution. Continuing, he referred to the proposed C. B. I. Publication on Silting of Reservoirs. The bases of the book were the two papers submitted by Rai Bahadur Khosla in 1941 and 1942. Those papers had been compiled into one and brought up-to-date. The second part of the book dealt with the investigtions that had been carried out with regard to Bhakra, Kosi and Mahanadi, and included data collected since 1942.

THE SECRETARY then enquired whether Mr. Hardikar, could contribute any data on silting of reservoirs in Hyderabad and the life of reservoirs.

MR. J. C. HARDIKAR said that he would try to send whatever was available.

THE SECRETARY asked whether Madras could contribute something.

RAO BAHADUR A. R. VENKATACHARI said that they had not collected any data so far.

RAI BAHADUR C. L. HANDA wanted the opinion of the house as to whether it was possible to carry out any model experiment on silting of reservoirs the phenomenon of sedimentation and as to what happened to the deposits over a period of time and what changes did take place.

DR. H.L. UPPAL said that they wished to get out a model of the Bhakra Dam on a scale of 1 : 25. The cost of this model was going to be something like seven or eight thousand rupees. He wanted to know whether any useful research could be undertaken to study the phenomenon of silting.

RAI BAHADUR GITA RAM GARG enquired as to how the rainfall was arranged for.

DR. H. L. UPPAL said that actually this model was contemplated for study on density currents and to study silt suspension. The model would be constructed soon. It would be about 100 feet long. The lake would be 900 to 1,000 feet wide, depth from the lowest bed would be 22 feet and normally it would be 10 feet. While he was in America he discussed about this kind of model with Mr. Savage and Mr. Eatten. Both of them were of the opinion that the model might give useful information regarding sedimental deposits, silting of tanks *etc*.

In answer to Rao Bahadur A. R. Venkatachari Dr. H. L. Uppal said that there was no vertical exaggeration introduced in the model.

The President expressed his opinion that this experiment might give useful results regrading density currents. The sort of silt distribution might be found out but not the sedimentation analysis. This was because the silt was brought by the river flow from all over its catchment.

Dr. H. L. Uppal said that the major difficulty was that in the area represented the settlement load might not occur correctly. So they had also an idea of representing four miles of catchment upstream, leave a few miles and then represent another few miles so that the whole upstream reach could be represented.

The President suggested that the whole reservoir was better taken as where it ended. The main slopes upsteram, and the main guages might be represented to find the load at each slope; this might give some useful results Different intensities of silt and flow might also be tried; that would be a very useful line of approach.

The Secretary wished that the East Punjab Irrigation Research Station might as well prepare a programme of their proposed experiments and then circulate it to the members and get their ideas and comments.

Rai Bahadur C. L. Handa said that they were very grateful for this suggestion and promised to adopt it.

Dr. N. K. Bose warned that one point had to be kept in mind in such experiments and that was the rate of settlement of silt in the model and in the prototype. In the case of tidal rivers too the same kind of difficulty arose. The rate of settlement increased in certain cases. It might be necessary to introduce some sort of change in the model proposed by Dr. Uppal. They used to make use of alum for sedimentation.

The President agreed to the proposal as the settlement affected the time and the size of the particle.

Dr. H. L. Uppal said that they had already considered about this and a detailed note submitted for getting the sanction for this model, contained about its use.

Rai Bahadur C. L. Handa referred to the work to be done by the Sub-Committee appointed for laying down stardard of classification of silt.

The Chairman (Mr S. A. Gadkary) considered the work of the sub-committee as of far reaching importance. Some type of silt might not have any appreciable effect on the turbines for 100 years while others might do the damage in 2 years' time. Grading of silt with respect to the material of the blade would be necessary and the velocity in the canal would also have to be taken into account to find whether that silt would reach the forebay or not. It might be necessary to experiment and see at various stages of velocity what particles were carried forward in suspension, and provide for accordingly. It should be possible to obtain a model turbine for experimental purposes. When the next big order was placed the suppliers should be asked to give a turbine of 3 h.p. for experimental purposes made of the same material as the prototype,

Rai Bahadur S. K. Guha enquired whether one would attach much importance to the material of the turbine.

The Chairman said that this was important in view of the fact the material employed in making turbines generally bore only trade names.

Dr. R. C. Hoon said that the point that the Chairman had raised was one of the aspects of the silt problem. The other aspect was the filling up of the reservoir with silt which was also to be considered. He desired that the sub-committee should appreciate that silt stations were out of the way places and that one could not have the same sort of accuracy at these places which one could have in the laboratories. Therefore, while making recommendations they might divide up the work in two parts, one that had to be carried out in the field and the other in the laboratories.

The Secretary suggested that all these points be taken note of by the sub-committee.

(ii) Capacity Surveys

PRELIMINARY NOTE

There was no item for discussion at the 1947 Research Committee Meeting under this head.

Recent Literature.

(1) Peechi Reservoir scheme, Cochin State—Chief Engineer, Cochin Government.

(2) Lang J. D.—The determination of storage reservoir capacity—State Rivers and Water Supply Commission, Victoria, Technical Bulletin No. 3, 1947.

(3) Inauguration of the King George VI Reservoir Metropolitan Water Board—Water and Water Engineering, Vol. 50, No. 622, December 1947.

(4) Shiam R.—Note on measuring storage capacity of reservoirs—United Provinces Public-Works Department, Technical Paper No. 10. Allahabad.

THE YEAR'S WORK

DISCUSSION BY THE RESEARCH COMMITTEE

There was no contribution and no discussion.

It was decided that the subject should remain on the agenda.

DISCUSSION BY THE BOARD

There was no discussion.

It was decided to retain the subject on the agenda.

4 C. Dams and Weirs

PRELIMINARY NOTE

The above subject was divided into four sub-heads in 1947, (*i*) Investigation, (*ii*) Design, (*iii*) Materials and (*iv*) Construction.

(i) Investigation

PRELIMINARY NOTE

There was no contribution or discussion at the 1947 Research Committee Meeting under this sub-head.

Recent Literature.

(1) Nevins T. H. E.—Earth dams—types and designs—Commonwealth. Engineer, Vol. 34, No. 5, December, 2, 1946.

(2) Warragamba dams—site selection and pipe lines—Commonwealth Engineer, Vol. 34, No. 7, February 1947.

(3) The Clark dam—investigations and design—Commonwealth Engineer, Vol. 34, No. 6, January 1, 1947.

(4) Sill R. T.—Exploring dam foundations—Western Construction News, Vol. 21, No. 8, August 1946.

(5) Harty V. D.—Site investigations for reservoirs and dams—The Surveyor Vol. 105, No. 2821, February 15, 1946.

(6) Walter, D. S.—Reclamation biggest earth fill dam—Engineering News Record, Vol. 134, No. 12, September 18, 1947.

(7) The Hirakud Dam Project—Mahanadi Valley development—Government of India, Central Waterways, Irrigation and Navigation Commission, June 1947.

(8) Preliminary memorandum on the unified development of the Damodar River. (abridged) Cetnral Technical Power Board, Simla, 1946.

(9) Incodel, the Interstate Commission on the Delaware river basin, a decade of planned progress 1936-1946—The Interstate Commision, Philadelphia, Pensylvania, 1946.

(10) Nevins T. H. F.—Investigation of dam sites—Commonwealth Engineer, Vol. 34, Nos. 10 and 11, May 1 and June 2, 1947.

(11) Harty V.D.—Site investigations for reservoirs and dams—Transaction institution of Civil Engineers, Ireland, Vol. 72, 1946.

(12) Multi-purpose development of Indian rivers—Science and Culture, Vol. 13, No. 1, July 1947.

(13) Musil, Dr. L., and Fischer Dr. E.—Situation regarding the construction of Arch Dams in Austria—Survey of the Arch Dams existing or under construction. International Commission on Large Dams, Third Congress Stockholm, 1948, C8.

(14) Observations during inspection of Dams in Service—February, March, April 1947. Concrete Laboratory Report No. C-356—U. S. Deptt. of the Interior, Bureau of Reclamation.

THE YEAR'S WORK

The following items were discussed at the 1948 Research Committee Meeting :—

(1) Whirlpools over the Sangam—Anicut across the Pennar.
(2) Engineering Investigations by Dr. K. L. Rao, Design Engineer Madras.

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(1) WHIRLPOOLS OVER THE SANGAM ANICUT ACROSS PENNAR[1]

ABSTRACT

Model experiments to study the formation of whirlpools upstream of the left bank of the Sangam Anicut and its effect on the stability of the anicut were taken up for study on model. A 3-dimensioned model to scale $\frac{1}{400}$ horizontal by $\frac{1}{100}$ vertical and a part-width model to scale $\frac{1}{200}$ horizontal by $\frac{1}{50}$ vertical were constructed. The part-model was developed from the pilot model, and the whirlpools were reproduced in the former. Describes the model.

A layout of the Sangam Anicut across the Pennar showing the margins and flank connections is shown in Figure 4 C. 1. The head sluice of the Kanigiri Main Canal is located on the left flank. There is a groyne with its top at shutter crest level, bifurcating the anicut and the undersluice portions. There is a protruding hill on the left margin a little ahead of the anicut. The flood water approaching the anicut rubs against the upstream face of the hill and is deflected

(1) Irrigation Research Station, Madras, Annual Report 1947, pages 22—26.

away from the margin. Owing to the presence of still pond downstream of the hill powerful whirlopools are generated commencing at the nose of the hill and extending in a line towards the left end of the anicut and crossing the divide groyne which gets submerged. During the 1946 floods,the divide groyne collapsed and a breach occurred to the anicut itself for a length of 400 feet commencing from a point 150 feet distant from the end left of the anicut. Model experiments were thereupon taken up to investigate whether the damages were attributable to some enxtent to the occurrence of whirlpools and if so to devise remedial measures.

For these studies, a rigid bed model with a slight vertical exaggeration was indicated. First, a three-dimensional model with a rigid bed to scale $\frac{1}{400}$ horizontal and $\frac{1}{100}$ vertical and reproducing the reach 5,280 feet upstream to 2,640 feet downstream was constructed. The model was intended to serve as a pilot model for the study of flow pattern alone. A flood discharge corresponding to 810,765 cusecs computed by Froudian scale ratio was passed and the water level elevations at points 53 feet upstream of anicut and 605 feet downstream of anicut were observed, these being the positions of gauges in the prototype. The tail-gate was operated to adjust the rear water level to equal the prototype value for that discharge. The afflux in the model was found to be 6·6 feet against 5·5 feet in the prototype presumably due to the lower discharge coefficient in the model. This being a distorted model equality of Froud's Number could not be simulated at various points in the model and hence the formation of standing wave downstream was incorrectly reproduced. Since the flow pattern upstream alone was considered important for the present study, the above deviation was ignored in the pilot model. The flow pattern was traced by wool threads. The depth being small no distinction could be made between surface and bed flow lines. It was decided to choose the stream line intersecting the anicut at a point 2,280 feet from the groyne as one boundary and to study the problem in a larger scale part-width model.

Accordingly a model to scale $\frac{1}{200}$ horizontal by $\frac{1}{50}$ vertical was designed.

DESCRIPTION OF THE PART-WIDTH MODEL

The model layout is shown in Figure 4 C. 2. It reproduced the configuration of the left margins as in prototype and on the right it was delimited by the path of the stream line traced in the pilot model. The reach 5,280 feet upstream to 2,640 feet downstream was included in the model. The bed was moulded rigid and plastered over in cement mortar. The supply to the model was gauge over precalibrated weir plates. There was a fore-bay with stilling racks, a tail-bay tail-gate and other usual appurtenances. Water level at points, 2,053 feet and 53 feet upstream and 1,605 feet and 605 feet downstream of anicut were tapped and communicated to gauge wells. The water levels in the wells were read by pointer gauges with 1/1,000 foot least count.

DISCUSSION OF MODEL PERFORMANCE

For arriving at the model discharge to be passed, a few preliminary computations had to be made. The discharge through the portion of the river represented by the model was assumed to be proportionate to the length of the anicut represented. Owing to the oblique flow from the flanks this asumption was not strictly correct but it was acceptable for the purpose of the present studies. Thus the discharge over the portion under study was taken as equal to 433,000 cusecs. (6·124 cusecs in model).

The value of he rugosity co-efficient in prototype was found out from the following relationships :—

$$Q = A.V. \quad \ldots \ldots \ldots \ldots \ldots \quad (4\ C.\ 1)$$

$$V = \frac{1 \cdot 486}{n} (d)^{2/3} (i)^{\frac{1}{2}} \quad \ldots \ldots \ldots \ldots \quad (4\ C.\ 2)$$

For A the average cross-sectional area in the reach 3,520 feet upstream was taken.

Q=8,10,765 cusecs flood discharge.

d=Hydraulic mean depth.

$$= \left(\frac{\text{Mean area}}{\text{Mean velocity}} \right)$$

i=3·8 feet/mile (observed in prototype).

The value of prototype 'n' worked out to 0·01613.

The value of 'n' in model was similarly computed. For this purpose a model discharge correspoding to 433,000 cusecs of prototype, computed by Froudian Law was passed.

$$\text{Thus } Q = \frac{Q_{\text{prototype}}}{b_r \times d_r^{3/2}} = 6 \cdot 124 \text{ cusecs.}$$

b_r=horizontal scale ratio.

=200.

d_r=vertical scale ratio.

=50.

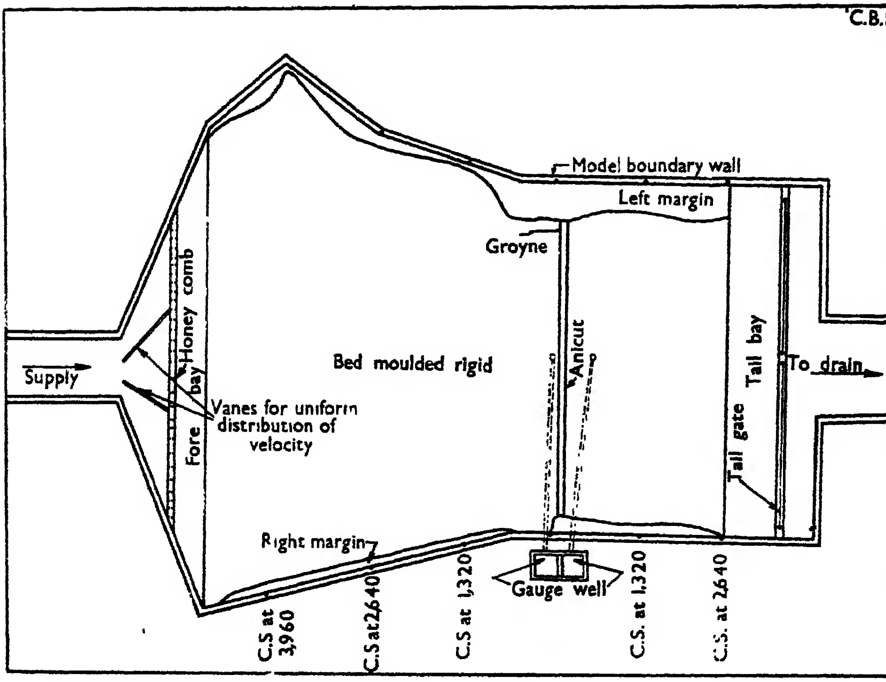

Figure 4C.1:-*Showing Sangam Anicut across the Pennar-layout of full model*

Scale 16 12 8 4 0 16 32 48 Hundred feet

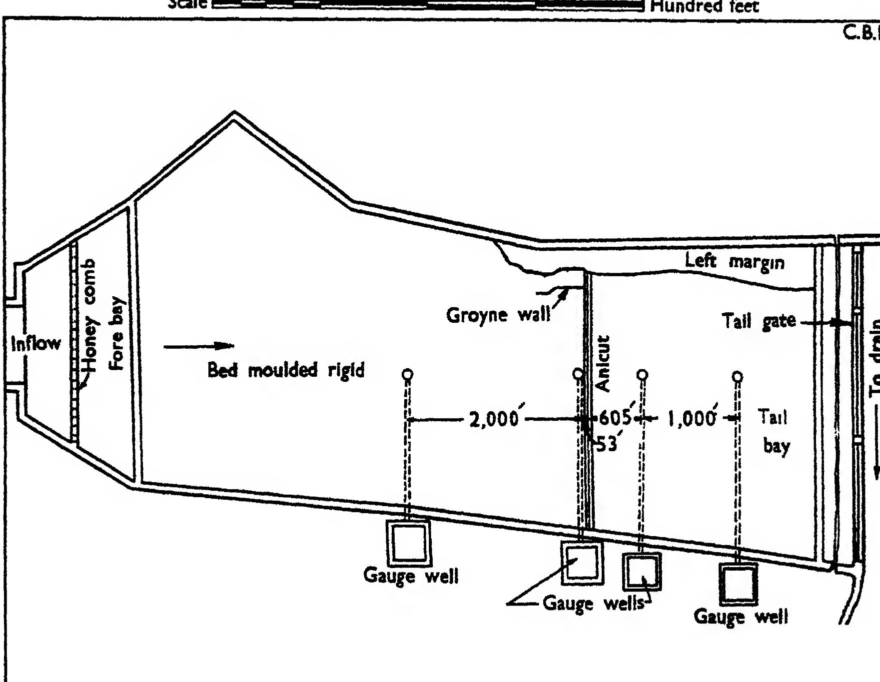

Figure 4C.2:-*Showing Sangam Anicut-layout of part model*

Scale 12 8 4 0 12 24 36 Hundred feet

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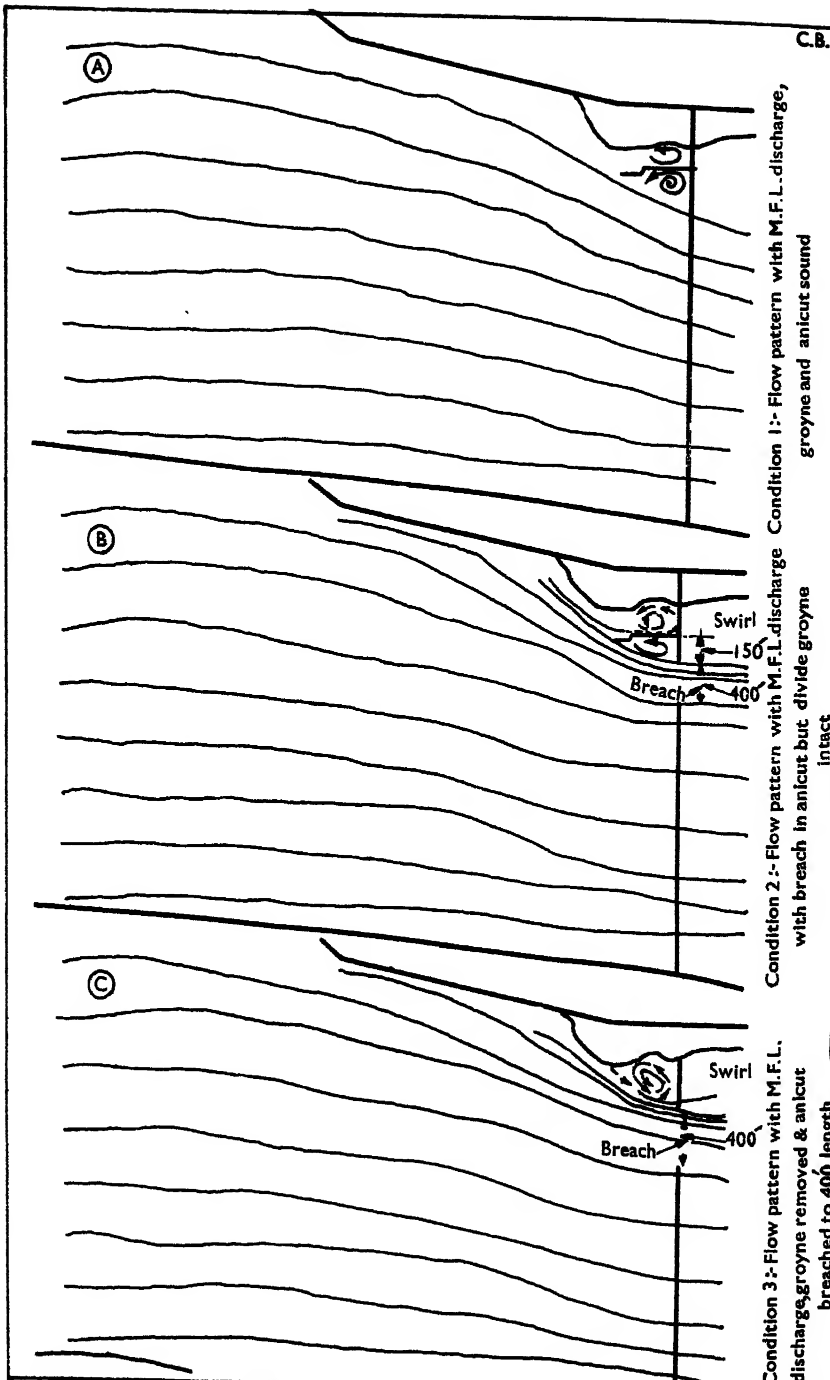

Figure 4C.3:- *Showing Sangam anicut part model* $\frac{1}{200} \times \frac{1}{50}$

Scale 400 200 0 400 800 1200 Feet

Reg. No. 2737 XDD (C) '50-1,210 (P.Z.O.).

This model flood discharge was passed and the tail gate adjusted to maintain a level of + 114·8 feet, at 605 feet downstream of anicut as observed in prototype. For this condition the surface fall was noted and the value of 'n' computed precisely in the same way as explained above for prototype. The model 'n' was found to be 0·01855.

The water-level upstream of anicut was +122·35 against + 120·6 in prototype and the surface fall actually attained in model was 13·2 feet/mile which represented 3·3 feet/mile taking vertical exaggeration of 4 into account, against a prototype surface fall of 3·8 feet/mile. The upstream depth in model was thus greater than what it should be and the surface fall was much flatter than the prototype conditions. Since the formation of whirlpool is mainly dependent on the flow pattern and the depth, the above feature had to be correctly simulated for proving the model. The heading up in the model was probably due to the lower co-efficient of discharge of anicut in the model. The correct upstream water level was, therefore, obtained by uniformly lowering the crest in stages. The surface fall in the model was different owing to incorrect simulation of friction as the following discussion shows.

The discharge ratio should follow the Froudian relationship since the flow passes over an anicut which acts as one of the controlling sections.

$$Q_r = b_r\, d_r^{3/2} \quad \ldots \quad (4C.\ 3)$$

where b_r=horizontal scale ratio.

d_r=vertical scale ratio.

Again the choice of b_r and d_r fixes the slope scale, *viz.* $i = d_r/v_r$. For simulating the frictional effects throughout the model the Mannings discharge relationship should hold.

$$\therefore\ V_r = \frac{1}{n_r} \cdot d_r^{2/3}\, i^{\frac{1}{2}}$$

$$= \frac{1}{n_r} \cdot d_r^{2/3} \frac{(d_r)^{\frac{1}{2}}}{v_r}$$

$$\text{Also } Q_r = \frac{1}{n_r} \cdot d_r^{2/3} \frac{(d_r)^{\frac{1}{2}}}{V_r} \times b_r\ d_r$$

$$= \frac{1}{n} \cdot d_r^{13/6}\, b_r^{\frac{1}{2}} \quad \ldots \quad (4C.\ 4)$$

Hence in this study the discharge scales *vide* Equations (4 C. 3) and (4 C. 4) should be simultaneously satisfied if the correct depth and surface fall should be simulated in the model.

$$Q_r = b_r\, d_r^{3/2} = \frac{1}{n_r} d_r^{13/6}\, b_r^{\frac{1}{2}} \quad \ldots \quad \ldots \quad \ldots \quad (4C.\ 5)$$

$$n_r = (d_r^{2/3}\ .\ b_r)^{\frac{1}{2}}$$

$$= \frac{(50)^{2/3}}{(200)^{\frac{1}{2}}} = 0{\cdot}9596 \quad \ldots \quad \ldots \quad \ldots \quad (4C.\ 6)$$

Actual '*n*' ratio making use of data computed in the beginning—

$$n_r = \frac{0{\cdot}01613}{0{\cdot}01855} = 0{\cdot}9356 \quad \ldots \quad \ldots \quad \ldots \quad (4C.7)$$

Thus friction was satisfactorily reproduced in the model.

Experiments.—The flow patterns were then traced for the M.F.L. discharge for the following conditons and are shown in Figures 4 C. 3 (A) to 4 C. 3 (C).

(1) Groyne and Anicut sound Figure 4 C. 3 (A).

(2) Anicut with a breach for a length of 400 feet starting from a point 150 feet from the left end Figure 4 C. 3 (B).

(3) Anicut with breached portion and groyne removed—Figure 4 C. 3 (C).

The whirlpools were predominant in all the conditions though they were most powerful in condition (3) owing to the concentration of flow through the breach.

Owing to the vertical exaggeration in the model the whirlpools too were more pronounced.

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(2) ENGINEERING INVESTIGATIONS

by

Dr. K. L. Rao,

Design Engineer, Madras

ABSTRACT

The investigations connected with planning and design of a dam generally occupied a long time. But this period can be considerably reduced if investigations are undertaken in a systematic manner and if the data required and experiments to acquire that data are clearly throughout in advance the main investigations for a masonry dam are briefly described in the note.

SURVEY

A complete survey of the dam site showing the axis of the dam abutments and other features must be prepared. By observations, Sun and Star, time azimuth of the dam axis must be determind. Latitudes and longitudes are to be marked. Lengths along dam axis must be test-checked two or three times, for a miskate found long afterwards will throw out many calculations.

The Area of the dam site shall be divided into squares of 200 feet or 100 feet by running lines parallel to the dam axis and at right angles to it.

The features of the dam, and all other observations can always be referred to these standard axes.

BOREHOLES

One of the earliest investigations is the determination of depth of sound rock below bed level of the river. Either Ingersal's Calyx drill or sullivans core drills can be employed to obtain cores of two inches. It is considered desirable that continuous cores should be taken 20 feet to 50 feet into the rock and the percentage of recovory determined. The idea of these cores is just to be sure that the rock is in compact mass and is not out of an individual boulder.

PENETROMETER TESTS

Where there is a great overburden of earth, penetrometer tests prove cheaper and quicker in determining the depth of rock. The test consists in driving one inch diamter rods fitted with conical shoes with a 100 lb. hammer falling through 30 inches. The weight of the hammer and its fall may be varied to suit the soil conditions. The number of blows required for a foot of penetration are noted and give an index about the relative compaction, of over burden soils and finally the depth of rock. Thus at Ramapadasagar Dam, the Penetrometer went through easily in a portion of the river bed for 90 inches while in another area with a similar overburden of pure sandy soil, it took a number of blows for a foot of penetration for a similar depth. It was easy to see the reason for this from the configuration of the river. The area in which the Penetrometer pierced easily represents the portion where there is considerable scour each year and the sand is, therefore, in a loose state. It is, however, to be noted that penetrometer observations must be confirmed in a few cases by boreholes.

PERMEABILITY TESTS

One of the most important tests that should be done in the investigation stage is with a view to find the permeability of the soil so that the water that will seep into the excavation can be computed. Unfortunately tests will not alone reveal this quanity. Laboratory determinations of permeability, while of use as giving an approximate idea, are not of much use as permeability in place is different from that of the disturbed specimens. Rough estimates

can also be made from curves if the mechanical analysis of soil is known. The best method is the " Pumping Test ". At important places, wells are formed of not less than 10 inches to 18 inches diameter. If the depth of overburden is not great a casing pipe of 15 inches or 6 inches more than the desired well diameter can be driven to the rock, inner casing with slots inserted to form a well of 10 inches diameter and the intervening space filled with gravel of 1/4 inch to 3/8 inch. The outer casing is withdrawn. If the depth is more, other methods must be investigated. Percussion methods coupled with jetting met with complete failure at Ramapadasagar Dam site. " Reverse Rotary Rig " developed in America offers an excellent alternative. The apparatus consists in a three feet diameter cutter attached to a hollow shaft. To start with clay is piled for a depth of four feet above the site and water is started on it. The cutter started and water pumped out continuously through the central shaft. All the time water is maintained at four feet above the ground level. This permits collapse of the sides of the hole though it is not lined. The small clayey particles are assumed to clog the sides of the holes. The hole is drilled very rapidly at one foot for a minute and after the required depth is reached time slotted casing is inserted and sides filled with gravel as before. I have seen a hole being made with this apparatus in a sandy river bed (in Northplat river Colorado) to a depth of 30 feet.

After the wells are formed deep well pumps are let down and water pumped out. Pilzometric wells 1½ inches to 2 inches diameter are formed in two directions at right angles to each other and radiating out from the well. Depth of water surface is noted as the water is pumped out. After some hours of pumping, e uilibrium conditions are obtained at which the water level remains constant in the well if a constant pumping is maintained. The water is measured by notches or measuring tanks. Value of permeability is then calculated from the observations.

Sometimes a reverse process is employed. Water is pumped into the well and similar observations kept.

The limitation of the pumping tests is in the fact that the yield may be different at other places due to difference in soils and the yield of the well will be different when a number of pumps are operating in the area.

Nevertheless pumping tests give valuable information. It is regrettable that no good and successful pumping tests have been made even in America, the few ones done at Nebraska, Dams' Dam being unsatisfactory.

INJECTIONS

If the over-burden is very permeable and considerable amount of water allowed to flow into the drainage there will be a danger of slides and unsuitable slopes to eliminate this danger, it will be necessary to adopt measures so that the inflow of water is less and slopes are stabilised. A number of methods are

suggested. Chief among these are, (1) Chemical injections (2) Bitumen injections known as shell Perm Process (3) Clay and cement injections. Of these the last is the cheapest and was said to have been carried out with great success at Genessiat Dam in France and Kentuckey Dam of Tannessy Valley Authority shell Perm has been employed successfully in Egypt in remodeling of Esna Barrage. The particular type of injections depends on the average grain size of the material and permeability.

Dr. Terzaghi strongly recommended Clay injections for Ramapadasagar Dam and suggested employing an Austriala expert for this. It is regrettable the investigation was not done so far. The cost involved was small compared with wealth of information it would have provided.

GEOLOGICAL MAPPING

The Geological Mapping of the area and classification of rocks in foundation and abutments should be done at an early stage. It is necessary, however, to realise that the surveys as such will not reveal the cemplete story. At Grand Coulee Dam-site inspite of best geological advice a fault was discovered only after opening the foundation. No dam has been given up so far due to the unsatisfactory nature of the rock. At Davis Dam inspite of best investigations during excavation the foundation rock for powerhouse and spillway was found to be so faulty that a million dollars are being spent in grouting operations.

RIVER DATA

It is necessary to obtain complete data of the river with regard to its evation discharge and silt content at the dam site and same distance below and above the dam site. It is true that the data to be reliable should be spread over a number of years but still a year's data will be helpful and far better than assumption.

COFFER DAM DATA

The diversion of the river during the construction period is always a major problem. If the water could be disposed off through side tunnels, it is well and good. But if the water has to be let down through a portion of the original river itself, a coffer dam has to be constructed to be safe against erosion and over flow. In the case of Ramapadasagar Dam, it assumes great importance. Data required for a successful design of the coffer dam has to be obtained from model and field tests. The hydraulic model tests may give an idea of the scour that may be formed and proper alignment of the Coffer dam to induce smooth flow. The field tests consist in driving the sheet piles and observing the depth to which they can be driven easily without injuring the interlocks.

SOIL TESTS

If the overburden is great, soil test must be done to determine the type of soil and know in advance the stability of slopes in excavation. In Ramapadasagar Dam, the soil tests are so important that the soils laboratory set up there and opened by Dr. Terzaghi is perhaps the best equiped in the whole of India.

WEATHER DATA

A record of temperature maximum and minimum of dry and wet bulb thermometer must be maintained at frequent intervals of the day throughout the year. The temperature of river water and its *pH* value must also be kept. The data is of considerable use in calculating the temperature of concrete inside the dam and design of cooling arrangements.

CONCRETE LABORATORY TESTS

A complete set of laboratory tests should be done on sand aggregate and cement that will be employed. The tests will include besides the routine tests, petrographic examination of aggregates to detect the presence of deleterious minerals such as opal and challcedany, determination of percentage of flaky and alongated material, soundness specific heat, conductor and diffusivity of aggregate and heat of hydration of cement. Tests for concrete must then be conducted. The most important is the design of mix for the mass concrete of the dam.

CHOICE OF MATERIALS

The chief problem confronting the Indian Engineer at present is whether (1) to use lime *surkhi* mortar and build the dam in masonry or (2) to use cement mortar and build the dam in masonry or (3) to use cement concrete entire for the dam. The considerations invloved in this seem to be (1) usage in the past (2) availability of lime in plenty (3) anxiety to show employment to the masons. But if the dams are to be built rapidly, cement concrete is undoubtedly the best. The rate at which it can be laid is 6 to 8 times as great per day as the rate of masonry construction. But concrete dams involved importing of foreign plant and cooling problems.

There is the question of cost to be taken into consideration. It is argued that dam, built with *surkhi*-mortar, even though has a bigger section, is cheaper. As against this, absence of criteria to ensure uniform mortar is a serious draw-back to the use of lime. Built with even indifference cement scores easily over the lime. Whatever might have been prejudices against cement in the past being definitely a national industry it should be employed even if it proves lightly costlier.

When cement is employed in concrete dam it should be of moderate heat type. That is, its heat of hydration should not be greater than 65 at the end of

seven days' compared with 80 to 90 of normal cements. It is easy to produce this type of cement. A little admixture of iron, an alteration in the relative proportions of lime and clay and finer grinding are all the alterations to be made in the normal process. Associated cement have agreed to do this.

AGGREGATES

The next important material to be considered is the aggregate. It will be economical as far as possible to use the aggregates at sites. In almost all the T. V. A. dams the rock at site has been crushed and used. While it is necessary to examine the rock for any undesirable minerals any type of rock can be used.

It is expected, there may be same trouble in obtaining natural sand at same dam sites. Perhaps crushed rock or mined and washed soil may have to be used as sand. In such cases modern bowl classifiers of Dorr Company should be considered.

ADMIXTURE

If cement is being used it is worth-while investigating whether it can be replaced by some admixture like *surkhi*. It must be admitted that no *surkhi* is mixed with cement in America. Nevertheless as *surkhi* will be less costly than cement in India investigation will be valuable. Some of the tests conducted at concrete laboratory at Madras show that there is no reduction of strength if cement is replaced up to 20 per cent by *surkhi*. The effects on durability and permeability have yet to be investigated.

CONCLUSIONS

Definite procedures followed in differnt provinces may be collected together and an instructive manual published by the Central Board of Irrigation for guidance.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. KUTTIAMMU introduced item (1).

DR. RAO introducing his paper on "Engineering Investigations" said that it was essesntial to carry out in a systematic manner the general investigations for the construction of any dam before designs were undertaken. These included a systematic survey of the dam site showing the dam abutments and other features. The standard axis should be fixed by dividing the area of the dam site into a grid say 200 feet by 100 feet. Referring to bore-holes he emphasized that a sufficient number of continuous cores should be taken say 20 feet to 50 feet into the rock and the percentage of recovery determined to ensure that the rock was in a compact mass and that the cores were not from an individual

boulder. He further explained that where there was a great overburden of earth it was cheaper and quicker to determine the depth of the rock by means of penetrometer tests as described in his paper. As regards permeability he said that the same could be worked out from pumping tests. He further explained by means of illustrations the working of the reverse rotary rigs and remarked that these methods were being extensively used for well irrigation in U. S. A. RAI BAHADUR BIJAWAT said that in connection with the construction of tubewells they were carrying out similar tests and further added that the students were being given training in this direction.

DR. RAO explained the limitations of the pumping tests and pointed out that so far no successful pumping tests had been carried out even in America. He then referred to the remaining items *i.e.* injections, geological maps, river data, coffer dam data, weather data, concrete laboratory tests, choice of materials, aggregates and admixtures.

RAI BAHADUR C.L. HANDA remarked that Dr. Rao had covered a very wide range in his short note. As regards art of drilling this was a technique by itsef, though it became more a matter for experience in the field. The process involved in each case could not be described as it largely depended upon the nature of the rock, type of drilling and the type of bits. He stressed that the driller and the geologist both should be at the spot in order to direct the drilling because it was of supreme importance to have an exact idea of the bore, the nature of permeability of the rock and further that the information regarding all these should be preserved in a detailed record. Referring to the investigation carried out at the Bhakra Dam site, he said that with the assistance of Dr. Nickell, and an American driller they had kept a satisfactory record of the drillings and that with the help of that data they had assessed the permeability of the rock and that Dr. Nickell even forecasted that the total seepage would be between 5 and 10 cusecs. He concluded by saying that apprentices should be attached with the drillers as they would thus be able to pick up the secrets of the trade in a short time.

DR. HOON gave a short survey of the soil exploratory work which he had done at the Hirakud Dam site and said that approximately 10,000 feet of the dam consist of about 4,000 feet of concrete portion and the rest earth. The latter was to be built in two sections *i.e.* in permeable and semi-permeable. The Soil survey consisted of making a number of pits in the borrow areas. The permeability of the soil had been found to be fairly low. Referring to the question of admixture of *surkhi* to cement mentioned by Dr. Rao, he said that he failed to understand what useful purpose that admixture would serve as *surkhi* was known to react with free lime of which there was little in cement.

RAI BAHADUR VENKATACHARI remarked that admixture of 20 per cent. of *surkhi* and sand with cement was used for the construction of the whole of Mettur Dam. According to him, *surkhi* being finer than sand helped in making the mortar denser as it was a better filler.*

*In two notes pertaining to the subject sent subsequently by Rao Bahadur A. R. Venkatacharya it has been stated that *surkhi* reacted with free lime released during hydratoin of cement and acts differently from fine sand suggested by some in place of *surkhi*

Mr. R. R. Handa enquired whether tests had been carried out to find out the strength and whether the economic side of this admixture had been looked into.

Rai Bahadur Venkatacharya replied that on the economic side there was an advantage though the question of strength had not been gone into. Further-more the Mettur Dam was not so high and therefore there was no likelihood of very high stresses developing.

Mr. R. R. Handa said that as the construction of a dam was to be in a river bed, sand could almost be had free whereas charcoal and fuel were required for the manufacture of *surkhi*. Even in 1920, the cost of *surkhi* was not less than Rs. 16 per hundred cubic feet in the Punjab. In these days it would be in the neighbourhood of Rs. 30. He therefore enquired as to why *surkhi* should be added to cement when sand would serve the purpose.

Sardar Bahadur Sarup Singh said that the use of *surkhi* in the Punjab did not prove a success.

Rao Bahadur Venkatacharya replied that the sand in Madras was very coarse and *surkhi*, which was finer, was used to replace cement as a binding element.

Mr. Mathrani said that the effect of the admixture of *surkhi* was to make the cement mortar slow-setting.

Mr. Gulhati drew attention to the admixture called sand-cement wherein a percentage of sand was ground with cement to the same fineness. This reduced the heat of hydration but did not reduce the tensile strength of the cement.

The Chairman (Rai Bahadur Venkatacharya) suggested that experiments should be conducted on the use of *surkhi* mortar and the economic aspect should also be gone into. He further added that the addition of *surkhi*, took the place of a filler and probably some sort of chemical action took place of which one was not aware of and it would, therefore, be better if detailed tests were carrid out.

Dr. Rao said that though *surkhi* was not being used in foreign countries as an admixture they were using other admixtures as good as cement. He added that in the tests that he had carried out they had used *surkhi* equal to one fifth of the cement and had found that the addition of *surkhi* upto 20 per cent. did not reduce the strength.

Rao Bahadur Joglekar said that figure 22 on page 23 of Madras Report showed the layout of the Sangam Anicut Model. The supply channel was so small compared with the width of the river and the Anicut that the inlet length appeared to be toally inadequate to reproduce correct flow conditions near the groyne. Either the width of the supply channel should have been more or the inlet length increased. This was important as the results obtained from this model were reproduced in a large part width model.

MR. KUTTIAMMU agreed with Rai Bahadur Joglekar's remarks that the supply arrangements were not quite satisfactory. But there were practical difficulties in improving the same and what was possible at site was done.

It was decided that the subject should remain on the agenda.

DISCUSSION BY THE BOARD

THE SECRETARY said that two items were discussed at the Research Committtee meeting (page 738). Item (2) ' Engineering Investigations ' by Dr. K. L. Rao aroused great interest at the meeting and then discussions centred on the strength and economic aspect of *surkhi* mortar.

RAO BAHADUR VENKATACHARYA suggested that experiments should be conducted on the use of *surkhi* mortars and the economic aspects should also be gone into. There was, however, no resolution.

RAO BAHADUR A. R. VENKATACHARYA suggested that Dewan Bahadur N. Govindaraja Ayyangar might like to say something about his experiences on the Tungabhadra and Rampadasagar investigations.

DEWAN BAHADUR N. GOVINDARAJA AYYANGAR said that he had been particularly associated with the construction of the Tungabhadra Project and the investigations of the Ramapadasagar Project. Of these the latter had many interesting features. The report on its investigations was ready to go to the press and would be printed shortly. The Ramapadasagar Dam is across the River Godavary near Plavaram village. There the width of the river is about 6,000 feet between the hills on either side Figure 4 C. 4 (a) and showing layout of west Coffer Dam. The maximum observed flood discharge of the river is 2·1 million cusecs, and the dam has been designed for flood discharging capacity of about 3 million cusecs. The bed level of the river is about +56 on the right side and zero on the left. The right side is practically above water level in the months of November to June. The left side carries the low water of about 1,500 to 2,000 cusecs.

The dam was to be of cement concrete using moderate heat cement and refrigerated with river water and by other methods. The spillway and the bulkhead sections are as in Figure 4 C. 4 (b). Over the crest of the spillway section were to be erected drum gates similar to those in the Grand Coulee, Boulder, Shasta and other dams in U.S.A. The full reservoir level will be +198 for the present, and +208 in the future.

The gross capacity of the reservoir at F.R.L. +198 will be 15·9 million acre feet.

The foundation for the dam would be very difficult to be laid. At the deepest point it would be necessary to go down about 240 feet below the bed level, to get at solid rock. A lot of borings and soil exploration at various depths below bed had been done. These borings and samplings were carried out not only to find out the nature of the bed rock but also the nature and characteristics of the overburden. There is a hill in the centre of the river with ex-

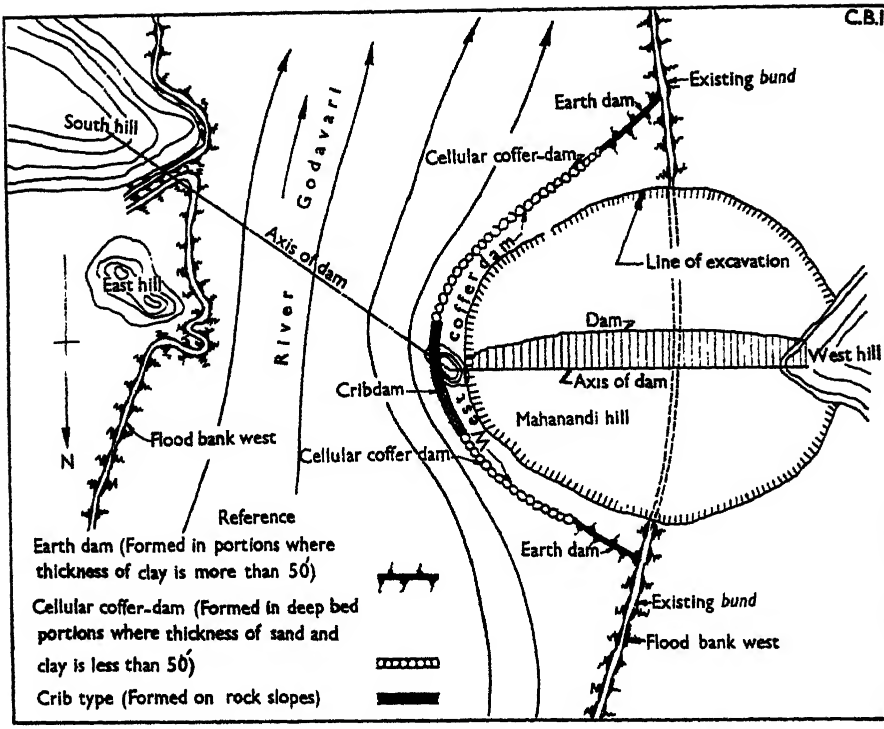

Figure 4C.4(a):- *Showing layout of Ramapadasagar west coffer dam*

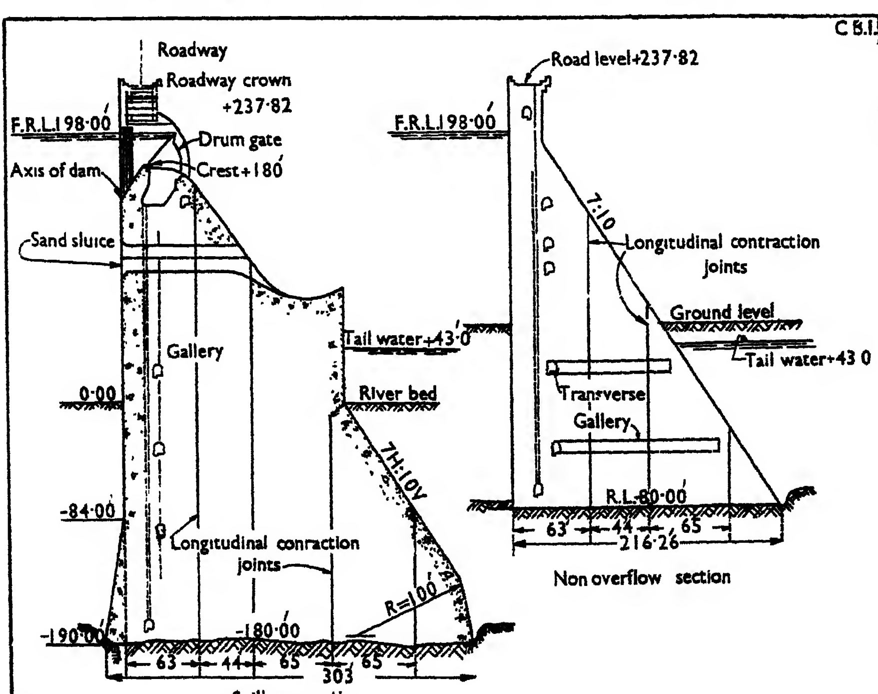

Figure 4C.4(b):- *Showing Ramapadasagar dam*

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posed bed rock ; therefore it would help to divide the dam into two halves for purposes of construction. From the point of view of anchoring the dam in the centre of the river, this central hillock would be very useful. It rises upto +133 and, therefore, stands above the maximum flood level of +90.

He added that the proposal was to divert the river first to the left side and commence excavation on the right side by forming a Coffer dam. The portion of the Coffer dam adjoining the central hill would be of masonry and it would not be difficult to construct it. Towards the right flank too, the difficulty in construction of the Coffer dam would not be great as there is a clay lens there in the over-burden at about 30 feet below the bed. But the central portions of the Coffer dam where the deep bed comes in formed by driving a series of inter-connected sheet pile cells to form what is called a cellular Coffer dam. The Coffer dam and the slopes proposed for the excavation for the foundations of the dam are illustrated in Figure 4 C. 4 (c). After the dam on the right side was raised to a sufficient level above the river-bed, the river would be diverted to the right side and the excavation on the left side proceeded with by forming a similar Coffer dam on that side.

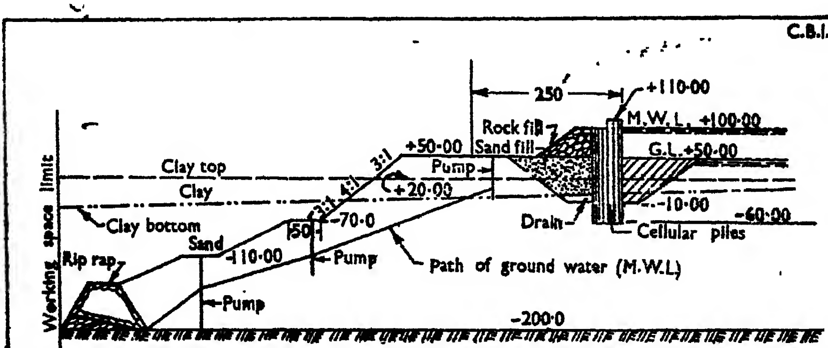

Figure 4 C. 4 (c) : *Showing line of excavation.*

The next main difficulty in the excavation of the foundation of the dam was the problem of enormous seepage that will be coming into the excavated trench through the heavy overburden consisting mainly of coarse sand. They had carried out certain pumping tests with 6 inches and 14 inches bore wells driven deep into the river bed and had taken observations for the curves of draw-down in the aquifer for different pumpages. The results of these tests have given a value of 40 feet per hour for K, the coefficient of permeability in Darcy's formula $Q=KAI$ where

Q=Pumpage in cubic feet per hour.

K=Coefficient of permeability in feet per hour.

A=Area of seepage in square feet.

I=Slope of the draw-down curve at the section considered.

Based on these results the seepage expected under the Cellular coffer dam was found to be the order of 1,000 cusecs.

How to manage this large amount of seepage and also how to remove the huge quantity of sand and earth quickly from the foundations of the dam were the most difficult problems to be faced. For such purposes some of the latest American methods of dam construction would have to be adopted. A good many excavators, drag lines and huge pumping equipments might have to be utilised if the work was to be successfully tackled.

He stated that other methods of reducing the heavy seepage from the foundation trench were also discussed in the report. For instance, in Amsterdam they were able to stabilise the sand and make it impermeable by injecting 'shellperm' a bituminous product. In South France they had utilised clay injections with a sheet of water glass in the centre. The third method that suggested itself was by soil freezing. One of these methods might have to be used during construction. The clay injection method would appear to be the most economic.

As regards canals system, there would be a canal on either side of the dam. The right side canal would be taken at its head through a tunnel 2,800 feet long. It would be 450 feet wide at head and 105 miles long till it would cross the Krishna River by means of an aqueduct. Part of the waters carried by it would be dropped into the Krishna for being utilised for growing the second crop in an area which already raises the first crop. The canal would be continued beyond the Krishna for about 90 miles length. The left canal would be 130 miles long and would tail into the Vizag Port.

Diwan Bahadur N. Govindaraja Ayyangar continuing said that the left canal would irrigate about 4·60 *lakhs* of acres and the right canal about 7·2 *lakhs* of acres besides the 7 *lakhs* acres of 2nd crop in the Krishna Delta. Including second crop under the new canals and in the Godavari and Krishna Deltas the total area under the project will be 2·75 million acres. He said that original designs for the dam were prepared in Madras. Six of the Madras Engineers were sent to America to study the latest designs adopted in America. The Madras designs of the dam and appurtenances had been checked by the International Engineering Company and had also been scrutinised by Dr. Savage, as Consulting Engineer.

Rai Bahadur C. L. Handa enquired whether the Madras Concrete Testing Laboratory had been set up.

East Punjab had received equipment for the setting up of a laboratory for Bhakra and Nangal Projects. They would like to have the description of equipment, how it had been installed and other details from Madras.

Dewan Bahadur N. G. Govindaraja Ayyangar replied that their laboratory was installed some time back and East Punjab could have any information or assistance from them.

Rai Bahadur C. L. Handa enquired whether any thought had been given to the possibility of constructing this dam in stone masonry with hydraulic lime mortar considering the present shortage of cement.

DEWAN BAHADUR N. GOVINDARAJA AYYANGAR in reply said that before suggesting the use of lime for such important constructions, lime should be standardised. One should ask oneself whether lime had been standardised, if not why not. There must be some reasons for not getting lime standardised even in these days of shortage of cement. He said that he had great respect for lime or lime *surki*. Stone lime is highly hydraulic by itself and there are places where hydraulic lime could be got cheaply but still cement has now-a-days come to be used on a large scale. This was because in the case of cement a certain standard had been set in the British Standard Specification, and many of the cement factories in India were today able to produce cement of better quality than even that set by the British Standard. He said he had been converted into the usage of cement only after he had seen the construction of Boulder and other dams in America. The people in India should learn to do things quick. In the R. P. S. they must aim at concreting for the dam at the rate of 10,000 cubic yards a day.

This, they must have to do with hammerhead or whirler cranes. Similarly the excavation of dam foundation might have to be done by powerful excavtors, shovels *etc*. One could not expect to complete this particular dam in 7 to 10 years, unless one resorted to these methods. His experience of the Mettur Dam was, that unless the Government completed the project early and tried to utilise its benefits for irrigation and power development the project would not pay quick dividends. Indian farmers especially in South India were still conservative and it would take sometime to persuade them to take to irrigations.

RAI BAHADUR R. R. HANDA enquired whether the Madras Government were doing the work themselves or through some foreign agency.

MR. J. C. HARDIKAR asked whether this project had been still an engineer's dream or had it received that blessings of the Government.

In reply DEWAN BAHADUR N. GOVINDARAJA AYYENGAR said that millions of cubic yards of earthwork and concreting had to be done. India had not till now developed so much mechanically. Our trained men were limited and those too did not know the required technique for such a big job. There were hardly a few power shovels here and there. In these circumstances he was of the opinion that it was impossible to do the dam except with foreign aid. There was no question about it.

Regarding the remark " engineers dream " he did not think that it was so. The Government had spent about 40 to 50 *lakhs* on the investigations of the dam and he did not think Government would throw away money, about 40 *lakhs*, for the sake of dreams. He was also of the firm opinion that engineers dreams were invariably such that they could always develop into solid concrete results. He sincerely hoped that the investigations that they had been carrying out would succeed and that was what Dr. Savage and many others had said.

The Chairman (Rao Bahadur A. R. Venkatacharya) said that with all these dams the shortage of rice in Madras was so great that even when the Rampadasagar would be completed, they still would have a deficit to face. So investigation of other projects had been going on. Proposals for two dams one across the Krishna, 800 feet above sea level and the other across Pennar had been put forward. The general width of the river Krishna is a mile and a quarter at the dam site it is about half a mile though in other places it is much wider. From this river they would take a canal through dividing ridge cutting down for a distance of miles and drop the water into the Pennar which is otherwise dry for the greater part of the year. The discharge of the canal would be of the order of 75,000 cusecs. When this canal is constructed it would provide water for about three million acres of rice crop.

In the case of Pennar the site of the barrage was in a gorge hardly two furlongs in width, where there was a depth of 100 feet of sand. He would try to shift the site of the dam a little higher. The flood discharge of the river would be about 100,000 cusecs. The canal was expected to irrigate about two million acres. At present aerial survey of the area was being taken. The Government of India had kindly arranged for the survey. Coming to the question of *surkhi* and cement, at the last Research Committee meeting there had been a long discussion. Some work had already been done in the laboratory regarding this. He did not consider that the reaction of *surkhi* and cement was so simple. As Mr. Handa had remarked there were two point about the addition of *surkhi* to cement. They in Mettur used 20 per cent. of *surkhi* in the cement. The main point was that cement and sand mortar or the cement concrete was affected by water in reservoirs. In Sweden and Norway too experience was that cement concrete and cement mortar were not good under conditions of submersion. Their experience was that upto a dilution of cement by 20 per cent. by *surkhi* the strength of concrete was not affected. While one could get the same strength and at the same time save some cement. Why not do it, especially now when cement was hard to obtain. Besides there was certain puzzolonic action between cement and *surkhi* which had been found to be beneficial to concrete. In the case of Lower Bhawani Project they would try to see how far they could dilute the cement content.

Mr. J. C. Hardikar expressed his doubts that in such a long canal as 150 miles would not evaporation losses be so great as to make it un-economical.

Rao Bahadur A. R. Venkatacharya said that since the discharge also was very high he did not think on the whole the losses would be great. It also depends on the discharge.

Mr. J. C. Hardikar said that assuming the same average losses as in practice, the canal irrigating lands 150 miles beyond head would not be economical.

The Secretary said that in the Punjab and Sind they had already canals of that length or even more.

Rai Bahadur C. L. Handa said that in multipurpose projects if they contain a high dam structure and also a system of lined canals, for instance for the Bhakra and Nangal, their experience was that the total use of cement was fifty fifty for the dam and the canals lining. Fifty per cent. of the total cement was required for high dams and fifty per cent. was needed for the canals. It was very important in view of the limited supply of cement to explore possible avenues of saving this precious material and in this connection the use of hydraulic limes had been tried. But there were two defects that had so far been noticed. One was the lack of standardization and secondly that research experiments as well as observations in the field had shown that it deteriorated in two ways, *viz.*, the surface gets roughened and the thing was not hard to the extent required.

Small scale tests had shown that if the lining of lime was treated with cement mortar layer varying from 1/8th inch to a maximum of 1/4th inch then one would have the advantages of cement in resisting erosion, as well as saving in cement. In this case they estimated that if they adopted this procedure for their projects, then they would save roughly about 90 *lakh* bags of cement *i.e.* on the entire canal lining they use a sub-layer upto six inches of *Kankar* lime concrete and gunite it on top ; the object of the guniting was to get extra resistance.

Rai Bahadur C. L. Handa said that he would be very obliged if Dewan Bahadur N. Govindaraja Ayyangar or Rao Bahadur A. R. Venkatacharya could give their opinion on the matter.

Dewan Bahadur N. G. Govindaraja Ayyangar said that for the dam cement was essential but for other purposes such as lining of canals etc., one might use anything, lime, hydrauic lime or cement.

For quick construction cement was helpful. While putting one substance over another one had to be careful. Lime requires more aeration than cement. Cement can set inside things while lime would require aeration. Also while placing one substance over another, if both were supposed to act together the question of differential expansion, had to be considered.

Rao Bahadur A. R. Venkatacharya suggested that guniting be done as soon as the lime concrete was finished. He also wished that five per cent. of crude oil be added to the mixture. The waterproofing and other advantages had been enumerated in an article in Indian Concrete Journal.

(ii) Design.

PRELIMINARY NOTE

The following items were discussed at the 1947 Research Committee Meeting :—

(1) Stress analysis by method of least work.

(2) Photo elastic studies.

(3) Variation of co-efficient of discharge, with reservoir levels for full opening of sluices in Krishnarajasagar Dam.

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THE YEAR'S WORK

The following items were discussed at the 1948 Research Committee Meeting:—

(1) Model investigations regarding the design of Nagwa Dam.

(2) Proposed design of Lower Sind Barrage at Kotri for effective sand exclusion from canals.

(3) Regulation of the Kotri Barrage—"still pond" or "semi open flow"—and consequent height of pocket gates.

(4) Uttiramerur Anicut across the Cheyyar.

(5) Stilling basin for Malampuzha overflow dam.

(6) Spillway dam of Manair Irrigation Project.

(7) hydraulic design of the Nangal Barrage.

(8) Model experiments on Nangal Weir.

(1) MODEL INVESTIGATIONS REGARDING THE DESIGN OF NAGWA DAM([1])

ABSTRACT

Describes experiments carried out and the results obtained for the protection of the bed dowstream of the dam.

Model investigations were conducted last year regarding the various types of weir crests for Nagwa Reservoir for maximum flood discharge. The profile of a weir finally adopted by the Chief Engineer, Eastern Canals for the said reservoir is shown in Figure 4 C. 5. This is similar to one already tested at U. P. Research Station *vide* Drawing No. 19 of Technical Memo. No. 17 with the only difference that the crest has further been widened to 10 feet to facilitate proper inspection of the weir. The co-efficient of discharge for the above profile is now found to be 3·07 only as indicated in a geometrically similar model to scale 1/25 this resulted in an increase of H. F. L. 856·00 for Inglis flood of 68,000 cusecs, the depth over weir crest being 11·0 feet now.

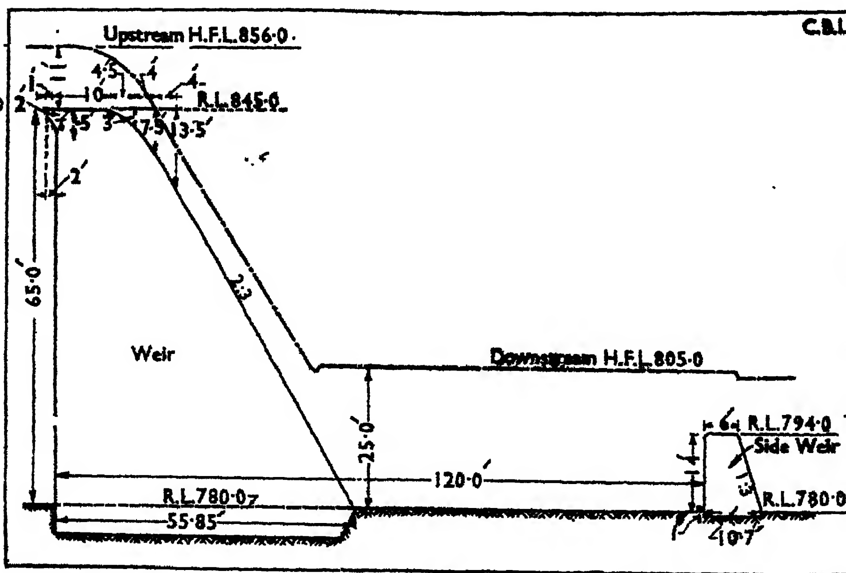


Figure 4C. 5 : *Showing cross-section of Nagwa weir and subsidiary weir.*

It was proposed last year that a 4·0 feet deep stepped cistern ([2]) 90 feet long with a row of staggered blocks would be adequate for the protection of weir downstream. The Executive Engineer, Mirzapur Canals, found it an expensive proposition to cut the rock at site to the desired level. As an alternative, a 14·0 feet high subsidiary weir at a distance of 64 feet was suggested by him *vide* Figure 4 C. 5.

([1]) United Provinces Irrigation Research Station, Report on Research progress during 1947, pages 59-61.

([2]) United Provinces Irrigation Research Station, Technical Memo. No. 17, Report on Research progress during 1946, Drawing No. 16, page 49.

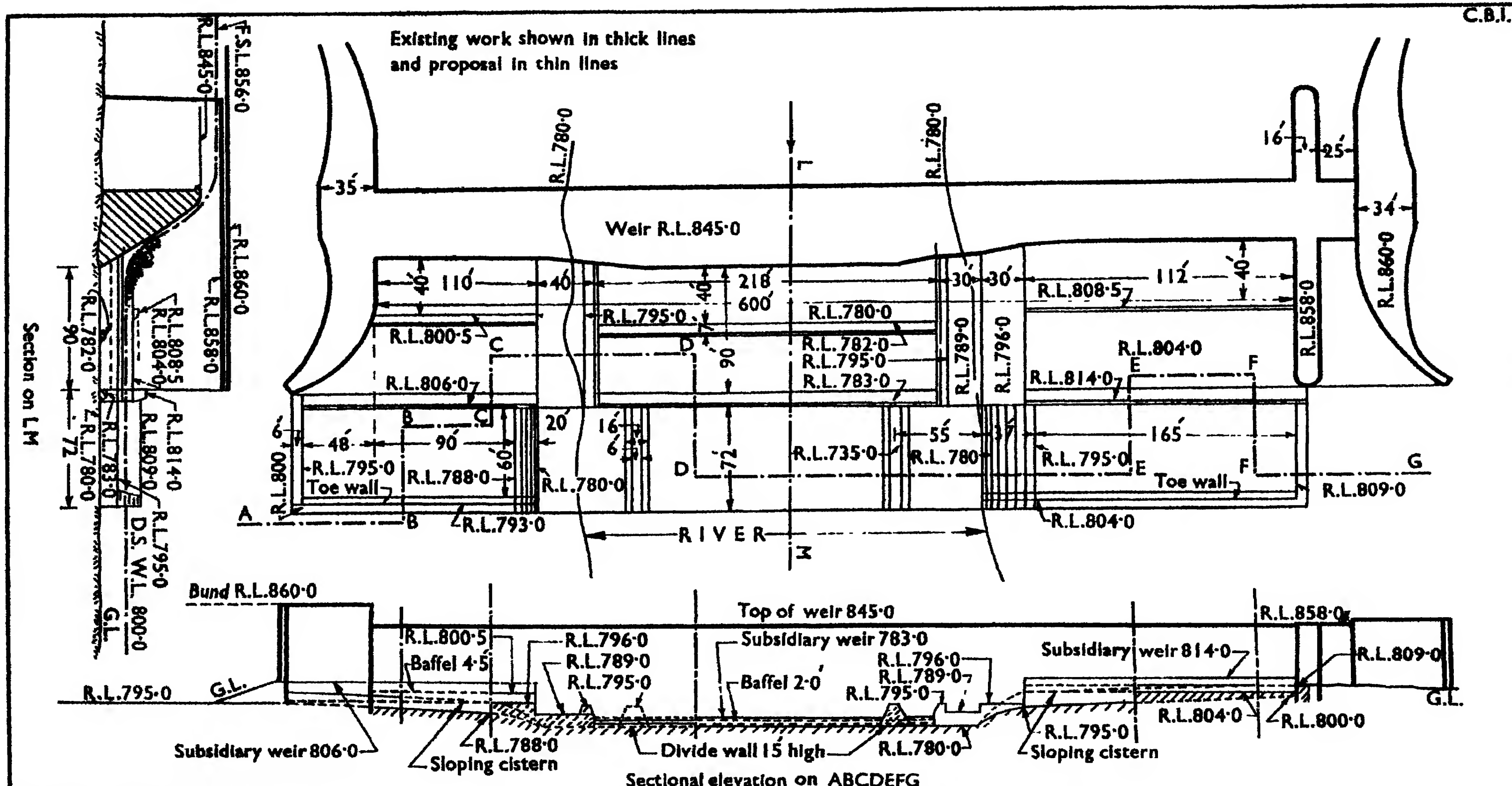
A part model of the weir to scale 1/25 was constructed in a 3·0 feet wide masonry flume. Pea gravel of size ¼ inch was used downstream for comparative study of scour *etc.*, as a heavier material was required to limit the scour to reasonable dimensions.

The 14·0 feet high subsidiary weir situated at a distance of 64·0 feet from the toe of the main weir gave 12·0 feet deep scour in a length of 80·0 feet. This excessive scour was due to the creation of a subsidiary fall at the low weir with no protection downstream. In view of the investigations carried out last year, subsidiary weirs of heights varying from 14·0 feet to 2·0 feet were tried. 3·0 feet high weir gave a minimum scour of 4·0 feet. Next the floor lengths were varied from 65·0 feet to 100 feet. The 90 feet long floor gave optimum results. But the action of the bed roller still persisted. A 2·0 feet high baffle located at a distance of 40·0 feet from the toe of the main weir considerably improved flow conditions, the scour was only 1·5 feet. However, the stepped cistern referred to earlier was definitely a better device.

The main weir is 600 feet long. Only 300 feet central portion is the deep channel. The natural rock levels on right and left banks are 14·0 feet and 22·0 feet higher respectively than the rock in deep channel. The problem to be tackled was to centralise the flow from the high banks on either side of the deep channel as smoothly as possible. This naturally involved a study of three dimensional flow. A geometrically similar model to scale 1/36, of the weir and the Karamnasa River was constructed. The river bed was made up of split boulders set in weak cement mortar to simulate the jetting rocks and provide for adequate roughness. 500 feet long river bed immediately below the weir was laid in peagravel to make a comparative study of the action of falling water.

No records exist for the floods in the past years for this river. Gauges were fixed at certain sites and during the last monsoon three observations for low floods were taken. Unfortunately, the intensity of floods this year had been very low, of the order of 11,000 cusecs only in comparison to those in the previous years when floods upto 63,000 cusecs had passed. The model was properly roughened to reproduce the above photo-type gauges. After this a stage discharge curve was worked out for the higher floods from the model itself. The maximum downstream full supply level was found to be 799·0 only against 805 stipulated by the Executive Engineer.

Figure 4 C. 6 shows details of proposed protection work downstream of the weir. These consist of 90·0 feet wide floor with 3·0 feet high and 300 feet long central subsidiary weir, and 10 feet high ones both on right and left flanks. The floor is stepped at the sloping banks on either side. A 60·0 feet wide and 5·0 feet deep sloping cistern is provided on both sides below the subsidiary weir at the banks to divert the flow smoothly on to the central channel. Two divide walls, each 14·0 feet high, are located below the banks to prevent excessive velocities in the centre. This arrangement has worked very satisfactorily for minimum bed and side erosion downstream.



Reg. No. 2737 XDD (C) '50-1,210 (P.Z.O.).

Figure 4C.6:- *Showing protection work downstream of Nagwa Weir*

Scale 100 50 0 100 200 Feet

The following recommendations have been sent to the Executive Engineer, Mirzapur Canals :—

(1) 90 feet floor length to be provided downstream of weir beyond its toe. Subsidiary weirs of height 10 feet, 10 feet and 3 feet respectively to be provided at the end of downstream floor at the right (164 feet), left (172 feet) and central (300 feet) portions. In addition baffles of height 4·5 feet, 4·5 feet and 2·0 feet to be provided correspondingly at a distance of 40 feet from the toe.

(2) The slopes on either side to be stepped. On the right two steps 40 feet wide and 7 feet high each and on the left three steps 30 feet wide each and 7 feet, 7 feet and 8 feet high respectively. Two cross wall 6 feet high each are to be provided at the lowest step.

(3) 60 feet long and 5 feet deep cistern sloping towards the central channel to be provided below the subsidiary weirs on either bank.

(4) Two divide walls 14 feet high each to be provided on 72 feet long floor in the central channel below the subsidiary weir at a distance of 55 feet each from the edge of either bank.

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(2) PROPOSED DESIGN OF LOWER SIND BARRAGE AT KOTRI FOR EFFECTIVE SAND EXCLUSION FROM CANALS (4)

ABSTRACT

This year's work mainly covers the experiments carried out for determining the salient features in the design of the barrage, such as :—

(a) Sills of Head Regulators ; (b) Design of the left and right pocket and the optimum number of spans; (c) Long pocket versus short pocket ; (d) Design of guide banks ; (e) Location of the left abutment ; (f) Left and right pocket divide walls ; (g) river conditions after the construction of the barrage; (h) Regulation of the Barrage and the necessity or otherwise of keeping the pocket gates hihger than the other barrage gates; and (i) Necessity or otherwise of paving the bed of the pockets.

THE MODEL

The model, which represented a reach of 8½ and 6½ miles upstream and downstream of the proposed barrage, respectively, had the following scale ratios :—

Length scale ratio ..	.. $\frac{1}{250}$
Depth scale ratio ..	.. $\frac{1}{50}$
Discharge scale ratio	.. $\frac{1}{62,500}$
Vertical exaggeration ..	.. 5·0
Surface slope exaggeration	.. 7·5 to 8·5 respectively.
	for $Q = 200{,}000$ cusecs.
	to $Q = 530{,}000$ cusecs.

(4) Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report, Technical, 1947, pages 116—133.

As a result of the model experiments the following conclusions were arrived at :

(a) *Sills of the Head Regulators* :

The sills of the Pinyari and Fuleli Head Regulators are to be maintained at R. L. 52·0 *i.e.*, flush with the pocket crest or six feet higher than the upstream pavement level (R. L. 46·0).

b) *Design of the left and right pocket and the optimum number of spans* :

Eight spans and four spans of 60 feet each were found suitable for the left and right pocket, respectively, from considerations of optimum scouring velocities, downstream cistern levels, shoaling in the pocket, and satisfactory V_R/V_P ratio which governs the curvature of flow.

(c) *Long pocket* versus *short pocket* :

The short pocket 637 feet long was found to be superior to the long pocket 850 feet long in respect of scouring operations, quantity of sand to be scoured and less time for scouring, reduction of Coffer dam work, and the control point (*i. e.*, divide wall nose) being nearer the barrage.

It is finally decided to have the left pocket 637 feet long with an open channel for Wadhuwah through the 25 feet common wall between the Fuleli and Pinyari Head Regulators, Fisadiwah offtaking upstream of the Pinyari Head Regulater through two hume pipes five feet diameter each.

(d) *Design of guide bank* :

The left guide bank as recommended is 5,900 feet long, concave throughout its length, so that it does not distrrt the existing conditions, avoids the deep water channel and allows water to follow it at all the river stages.

The right guide bank is recommended to be 6,650 feet long, convex concave in alignment.

(e) *Location of the left abutment* :

The left abutment is decided to be 325 feet from the toe of the Jamshoro *bund*, so as to have a short guide bank, less distortion of natural curvature, minimum reduction of waterway during the first season's Coffer dam work and less difficulties during construction of the abutment.

(f) *Left and right pocket divide walls* :

(i) The left and right pocket divide walls extending up to the upstream abutment of the Fuleli Feeder and the right bank Feeder respectively, were found better than the shorter divide walls.

(ii) It is also recommended that the apron protection round the noses of divide walls be laid in a cup-shaped pattern conforming with the extent and depth of the natural scour holes.

(iii) As the concrete blocks 4 feet × 4 feet × 3 feet (proposed by Sind) are not expected to settle uniformly, it was recommended that the apron be laid with 'two—to three--men' stones.

(3) REGULATION OF THE KOTRI BARRAGE—"STILL POND" OR "SEMI-OPEN FLOW"—AND CONSEQUENT HEIGHT OF POCKET GATES ([5])

ABSTRACT

As a result of model experiments and working experience of the Sukkur Barrage, the design of the Kotri Barrage was based on the assumption of "still pond" regulation up to the highest likely river discharge of 565,000 cusecs requiring the Pocket gates to be four feet higher than the other Barrage gates.

A few quick tests indicated that "semi-open" flow regulation was quite as good as "still pond" in the Kotri Barrage design in respect of sand exclusion, or discharges higher than the "still pond limit" (384,000 cusecs) —*i.e.* the discharge at which the Barrage gates would be just out of water with "still pond regulation" and pond level of R. L. 83·4. The chief merit of "still pond" regulation is that it assists naturally favourable curvature of flow in improving exclusion, where, for any reason, "semi-open" flow is equally effective, there are several good reasons to prefer it.

Further consideration and more detailed model tests were, however, necessary in view of the experience at the Sukkur Barrage where it was found necessary to raise the right Pocket gates by adding pieces at top in spite of the Pocket gates being originally 3 feet higher than the River gates; because it is considered inadvisable at Sukkur to open the Pocket gates at any stage.

A detailed comparison of the conditions obtaining at Kotri and Sukkur Barrage explains largely the reason why restricted 'semi-open' flow regulation is suitable at Kotri *though not* at Sukkur (right bank), thus :—

(*a*) Assuming curvature effect the same exclusion is governed by the ratio $V_R/V_{R_O} \div V_P/V_{P_O}$ or, for equal depths, by the ratio V_R/V_P (where V_R, V_P are the actual velocities and V_{R_O}, V_{P_O} the regime critical velocities on the river and Pocket sides of the Divide wall). This ratio for the Kotri Barrage is 1·74 times that at Sukkur, showing exclusion at Kotri is far superior to that at Sukkur even if curvature were the same at both.

(*b*) Actually, however, curvature is more pronounced at Kotri than at Sukkur, being intensified by the central dividing island at Kotri.

(*c*) The length of the Pocket at Sukkur is *over three times* as long as at Kotri. The longer pocket is worse for sand-exclusion (the control, *i. e.*, the divide wall nose, being further away) as well as for scouring at its entrance ; during scouring operations with a higher Pocket bed at the entrance at Sukkur the actual V_R/V_P ratio may be less than the calculated value (1·67) and the relative advantage of Kotri design would be greater.

([5]) Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report, Technical, 1947, pages 134-143 (Summary).

(*d*) At the Sukkur Barrage, the long separating walls between adjacent head regulators provide a considerable inactive region within the Pocket causing shoals. These shoals and still-water pockets within the Pocket adversely affect exclusion making 'semi-open' regulation inefficient.

Inspite of these comparatively unfavourable conditions, sand exclusion in the left Pocket at Sukkur is effective with 'semi-open' flow (restricted up to 4 feet gate openings) as both model experiments and experience have shown, in fact, the shoaling in the Pocket has reduced considerably since 'semi-open' flow regulation is in vogue at Sukkur at river discharges exceeding 350,000 cusecs, since the year 1943 at the repeated suggestion of this Station.

If restricted 'semi-open flow' regulation is by experiments and experience found adequate for the Left Bank Canals at Sukkur the same should also prove efficient at Kotri.

The Consulting Engineer to the Government of India when consulted on this issue, expressed the opinion that he would favour keeping the Pocket spans higher by a couple of feet *but no more*

This proposal has been carefully investigated on the model with a view to answer the following questions—

(*i*) Should the design be based on 'still-pond regulation' for all river stages; if not, to what river stage will 'still-pond regulation' be necessary and sufficient ?

(*ii*) Is it desirable to keep the Pocket gates two feet (or more) higher than the river spans gates or can this be avoided in any way ?

Obviously, the higher Pocket gates would be most needed after accretion upstream and downstream of the Barrage. The first experiments, directed to determine the 'still-pond' limit with Pond level R. L. 68·0 and four feet accretion upstream and downstream, showed that all the Barrage gates would be just open, pocket gates shut, for —300,000 cusecs. This agreed with the discharge worked out by calculations and so the lower limit of the range of discharge to be tested for optimum regulation was fixed at 300,000 cusecs. The upper limit was fixed at 500,000 cusecs from a study of the discharges expected after the contemplated further withdrawals in the Punjab take place. The maximum discharge for any five-day period, in the eleven years (1931-41) examined, after full development of the Indus Basin, ranges from 285,000 to 416,000 cusecs, with an average of 368,000 cusecs. Allowing for one exceptional year in every ten and for certain of the contemplated withdrawals being postponed or given up, 'semi-open' flow and 'still pond' regulation were compared over the range 300,000 to 500,000 cusecs.

The following four phases of regulation were critically studied at the discharge stages, 300,000, 350,000, 400,000 and 500,000 cusecs under all possible post-Barrage conditions downstream of the Barrage (*i.e.*, with retrograded, mean and accreted specific levels, downstream of the Barrage) :—

(*a*) Still-pond ;

(*b*) *restricted* ' semi-open ' flow so as not to overtop pocket gates of given heights ;

(*c*) unrestricted semi-open flow so as to get as low a water level upstream as possible ; and

(*d*) free flow.

The following Model data were collected in each case :—

(*i*) For each of these discharges, exclusion was tested by the usual method of bed seeds. In the quick tests carried out previously, bed seeds were dropped at cross-section mile 5½ ; but this time seeds were dropped at cross-section mile 5 (*i.e.*, half a mile lower down) so as to have a higher factor of safety in the results showing ' semi-open ' to be as good as ' still pond ' even for the bed material very near the left bank of the river at cross-section 5. (If bed seeds were thrown from a higher section, more were excluded under any regulation). The procedure followed was to drop a fixed number of seeds three times and then find the mean percentage of seeds entering the canals for conditions (*a*) to (*d*) as set out above.

(*ii*) Percentage of discharge in the left and right arm channels.

(*iii*) V_R/V_P ratio at the nose of the divide wall.

(*iv*) Difference in water level between the left and right arm channels with equal gate opening.

(*v*) Afflux in the left and right channels *i.e.*, difference in water levels upstream and downstream of the Barrage.

The experiments showed that the conditions at Kotri were better than at Sukkur and the results obtained in the model confirmed that *restricted* ' semi-open ' flow is certainly as good as ' still pond '. The model, however, showed that even with restricted ' semi-open ' flow regulation, it is necessary to keep the pocket gates at least two feet higher than the Barrage gates, *i.e.*, top at R. L. 71·0.

The model showed that, though designed for 25 per cent., less than 20 per-cent., of the total river discharge passes down the right arm channel *without regulation* between the two arms ; but this is more than sufficient for efficient exclusion of sand from the Right Bank feeder ; and the more the discharge going into the left arm, the easier becomes the exclusion from the Left Bank feeders. It may, however, become necessary to force the designed 25 per cent. down the right arm by regulation, for a time, so as to flush out the right arm channel of the Barrage. Such regulation would require a considerable difference in the water levels between the two arms of the river and, at that stage, there is the possibility of the left Pocket gates being overtopped, even if they are two feet, higher than

the Barrage gates. Scouring the right arm upstream of the Barrage may, however, result in sanding the right arm channel on its downstream side; because the increase in sand charge is likely to be disproportionately greater than the increase in the discharge. It will be seen from the graph of distribution of discharges that the natural percentages of discharge in the right arm do not exceed 14 per cent. and 20 per cent. in the pre-Barrage and post-Barrage conditions, respectively, and it is advisable to depart artificially as little as possible from these natural conditions.

It was indicated that silting is likely to occur near the right bank if the right bank pitching is more or less ineffective. Due to model limitations the model is not expected to show the silting *quantitatively* but it is apparent that silting would occur in the triangular portion upstream of the right arm barrage somewhere along the contour R. L. 55, but then a narrow channel would continue to be alive around the central island by virtue of the scour developed by the flow attracted by the island. At such a stage, it may be necessary to flush this channel; the gates must be such that they are not overtopped by reason of the high afflux required to flush the channel. Experiments carried out with $Q \equiv 5$ *lakh* cusecs showed that an afflux of 1·8 feet with upstream left pond water level of R. L. 75·4 was required (under the worst conditions obtained after accretion) to pass 26 per cent. of the discharge down the right arm. As, at this stage, the left Barrage arm remained ponded, which counteracted the curvature and increased the cut-off in the left arm gates it was not possible to open the left Pocket gates by more than two feet without adversely affecting exclusion. Under such conditions, the necessity of having the left Pocket gates four feet higher would be felt.

There is not the same necessity for having the right Pocket gates more than two feet higher than the Barrage gates.

RECOMMENDATIONS

(*a*) *Pocket Gates.*—If the top of Pocket gates are kept four feet higher from the start, there is the danger of the Pond level being unnecessarily raised or 'still pond' being maintained where 'semi-open' flow would be just as good, with the attendant disadvantages mentioned. Hence the top of pocket gates should be initially kept only two feet higher *with provision* to enable the left Pocket gates being subsequently raised, if required, by another two feet.

(*b*) Experience at Sukkur and experiments at Poona have shown that unfavourable curvature cannot be reversed by 'still-pond' regulation; but where there is favourable curvature, still-pond regulation assists exclusion, particularly at low stages of discharges—when the curvature is counteracted by

the heading up over the natural water levels and when the crucial ratio V_R/V_P governing exclusion is low. The recommended regulation is, therefore, based on the following *desiderate* :

(*i*) " Still-pond " regulation for as long as possible for low and medium discharges during the season ; and

(*ii*) the upstream water level to be maintained as low as possible by avoiding unnecessary heading up.

The experiments have, however, shown that it is necessary to specify different regulations for different discharges in relation to downstream as well as upstream water levels.

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(4) UTTIRAMERUR ANICUT ACROSS THE CHEYYAR (*)

INTRODUCTION

In Figure 4 C. 7 of the proposed Uttiramerur Anicut across the Cheyyar River as designed originally. The anicut was to be 1,080 feet long of which 880 feet had to be with a high co-efficient profile with crest at +234·14 and 200 feet will be with two feet falling shutters resting on flat crest at +232·14. The Chief Engineer ordered that model experiments should be carried out to examine the adequacy or otherwise of the protective works downstream of the anicut and to reduce the cost of construction as much as possible.

MODEL

A part model of the anicut to scale 1/18 was accordingly constructed. The model layout is shown in detail in Figure 4 C. 8. As may be seen therein the model of the anicut was fitted between two parallel walls six feet apart. This represented 108 feet of the prototype, the central 1 foot=18 feet being the falling shutter portion of the anicut and the rest being the high co-efficient profile overflow weir. The six feet wide flume was 26 feet long + 2·5 feet deep and was provided with a trolley gauge arrangement by means of which water surface elevations and scour contours could be traced correct to a thousandth of a foot. Water supply was obtained from the static tanks controlled by sluice valves and measured over a standard standing wave flume before admission to the model. A tail bay and tail gate were provided for adjusting downstream levels. Downstream of the model sieved sand passing through 18 and retained on 24 mesh per inch was packed to a depth of 1 foot and top levelled to correspond to + 229·14 in prototype. In all the experiments where scours were measured the period of run was one hour each time.

(*) Irrigation Research Station Madras, Annual Report, 1947, pages 34-45.

EXPERIMENTS

First series.—Eight experiments were carried out on this model in the first instance. In experiments 1, 3 and 5 the upstream water level was kept just at crest level, the falling shutters were removed and the portion covered by the falling shutters alone was allowed to discharge. In this condition no bed scour or flow condition was noticed. In all other experiments overflow over the high co-efficient portion also was reproduced in addition to the flow over the central portion. In this case shooting flow and harmful scours were seen for conditions of high afflux. The model was worked with and without the second and third aprons. It was also worked (1) without the staggered blocks (2) with staggered blocks 2 feet × 2 feet × 1 foot as shown in the original design and (3) with the height of staggered blocks increased to 2 feet. For an afflux of 3 feet there was no proper standing wave in any of these and scours as deep as 7 feet were formed. The design was obviously defective and it became evident that the simplest method of improving it would be to have a stilling basin arrangement in rear of the weir which would be designed so as to hold the standing wave within a short length of apron.

REVISED DESIGN

The profile of the weir was, therefore, slightly altered and a bucket and stilling pool were provided and this design was taken up for test. The design as finally evolved is furnished in Figure 4 C. 9. It consists of the weirs with and without falling shutters as before. On the downstream side a bucket with toe at +228·14 and radius 4 feet is provided. In continuation of the bucket is a stilling basin 5 feet 9 inches long and invert at +228·14 for the high co-efficient weir portion and a Schoklitch Stilling Pool 10 feet 3 inches long and invert at +227·14 for the portion with falling shutters. The upstream aprons are unaltered. On the downstream side the second and third aprons are eliminated and the staggered blocks are located at the end of the first apron. This design gave a good standing wave for the extreme conditions of discharge and afflux and very little scour. How the design was arrived at is described below.

EXPERIMENTS

Second Series.—In this 14 experiments were carried out. These are listed and explained in Table 4 C. 1. A reference to the statement will show how the second and third aprons could be eliminated, how the various baffle walls tried were unnecessary and how two rows of staggered blocks at the end were superior to having a simple deflector or Rehabock's dentated sill at the same place. For the overflow portion the stilling basin was sufficient to ensure a proper standing wave for all conditions of flow with high afflux. At the falling shutter portion a similar cistern could not suffice and hence the Schoklitch's Pool. The staggered blocks at the end dissipated any surplus energy left in the water issuing beyond the standing wave.

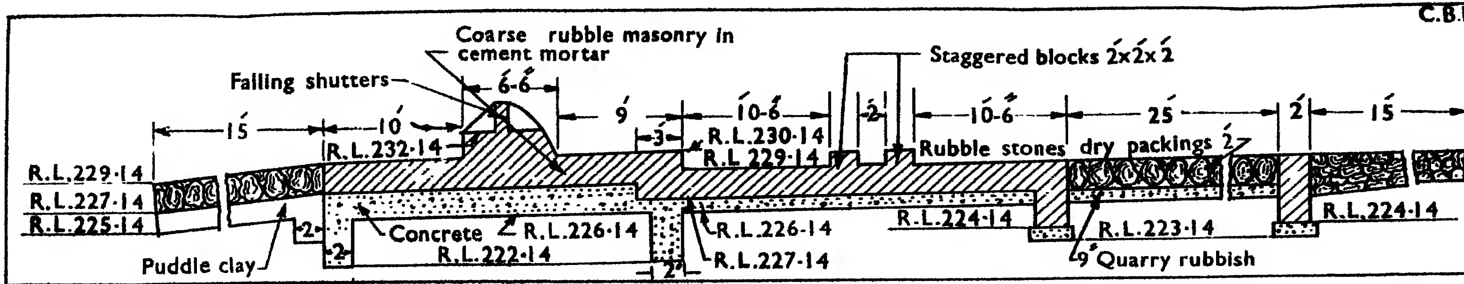


Figure 4C.7 :- *Uttiramerur Anicut -Design as proposed in the sanctioned estimate*

Scale 5 2·5 0 5 10 15 20 25 30 Feet

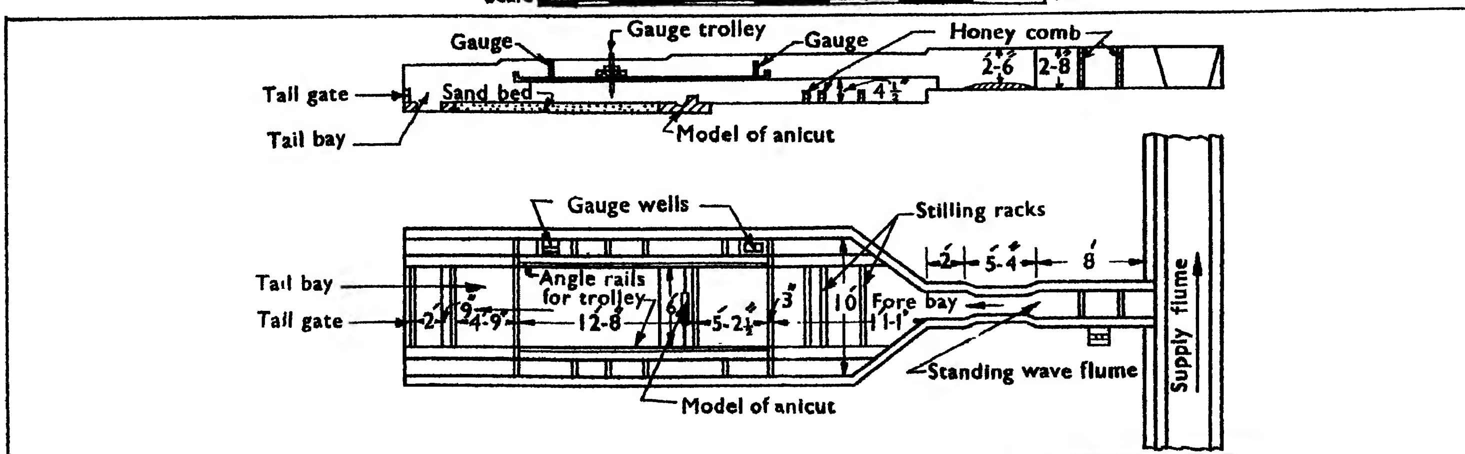
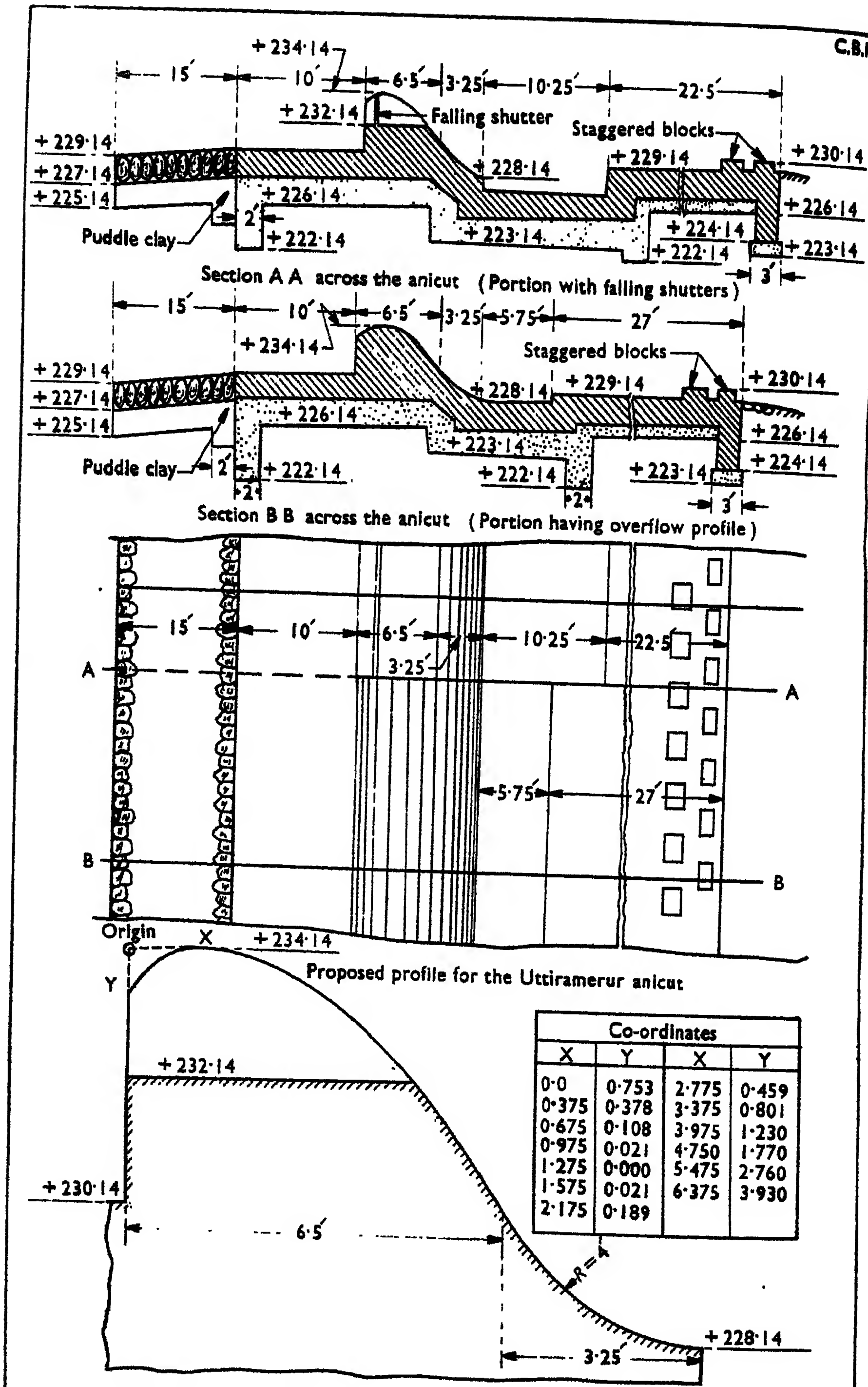


Figure 4C.8:- *Layout of part model of the Uttiramerur Anicut*

Scale 5 2·5 0 5 10 15 20 25 30 Feet

Reg. No 2737 XDD (C)' 50 - 1,210 (P.Z.O).



X	Y	X	Y
0·0	0·753	2·775	0·459
0·375	0·378	3·375	0·801
0·675	0·108	3·975	1·230
0·975	0·021	4·750	1·770
1·275	0·000	5·475	2·760
1·575	0·021	6·375	3·930
2·175	0·189		

Figure 4C.9 :- *Showing design of Uttiramerur anicut proposed after model experiments*

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TABLE 4 C.1

Serial number 1	Experiment 2	Dissipator arrangement 3	Up-stream water level 4	Down-stream water level 5	Dimensions and levels 6	Remarks 7
1	9	Stilling basin and baffle with crest + 230.64 and 86′ 9″ apron.	1′ over crest.	3′ below crest.	Stilling basin has invert + 228·4	Very little scour.
2	10	Do.	3′ over crest.	1′ below crest.	Bucket of 4′ radius and length 5′ 9″ level of apron + 229·14, baffle wall 2′ 0″ wide.	Standing wave over the baffle and high secondary wave within 1st apron scours 3′ deep along. Apron appeared unnecessary.
3	11	Same as above with apron length reduced to 59′ 9″.	Do.	Do.	..	Same as above. A further reduction in apron length possible.
4	12	Same as above with apron 32′ 9″.	Do.	Do.	..	Apron length all right. S. W. high. Reduction of baffle height suggested.
5	13	Same as above, but top of baffle + 230·64 for falling shutter portion and + 229·89 for the left and no baffle on the right.	Do.	Do.	..	No baffle necessary for high co-efficient weir portion. Stilling basin not all right for falling shutter portion.
6	14	Same as above, but baffle only for falling shutter portion. Also training walls at both ends of this.	Do.	Do.	Baffle and training wall 1′ 6″ wide	Even here S. W. appeared high for the falling shutter portion.
7	15	Schoklitch's stilling pool and baffle (+230·14) for falling shutter portion. Rest as for experiment 14.	3′ over crest.	1′ below crest.	Schoklitch's stilling pool has invert at + 227·14 and length 10′ 3″.	Standing wave high and scour 4 feet.
8	16	Same as above but baffle removed.	Do.	Do.	..	Standing wave all right for full length scour 3 only; design good.
9	17	Same as above with Rehbock's dentated sill at end.	Do.	Do.	Rehbock's dentated sill 1′ 10½″ + 0′ 9″.	Scour reduced to 2′ in both cases. To make a choice fresh experiments with exaggerated head of discharge suggested.
10	18	Same as above, but two rows of staggered blocks instead of Rehbock's dentated sill.	Do.	Do.	Staggered Blocks 2′ × 1′ 6″ × 1′.	
11	19	Same as per experiment 16.	4·7′ over rest	1·7′ Over rest	..	
12	20	Same as per experiment 19 two rows of staggered blocks.	Do.	Do.	..	Of the three sills tried the system of staggered blocks gave the least scour.
13	21	Same as experiment 19 Rehbock's dentated sill.	Do.	Do.	..	
14	22	Same as experiment 19 deflector.	Do.	Do.	Deflector 2′ 9″ × 9″	

Experiments 3 and 4.—A vertical sill 4½ feet wide × 5 feet deep was then introduced at the end of the basin: the basin was thus 15 feet long × 5 feet deep, having an upstream bucket of 20 feet radius. No pucca apron was provided downstream of the sill. The S. W. and scour profiles are furnished in Figure 4 C-11 (B). Standing waves were formed for all conditions flow. These rose above the tail water level and diving down gave rise to secondary waves. There was scope for improvement in the design.

Experiments 5 and 6.—The sill was therefore given a batter of 1 : 1 on the upstream side. This showed a slight improvement in the shapes of standing waves and in the scours. Please see Figure 4 C.11(C).

Experiments 7 and 8.—To see if further improvement could be effected the 1 : 1 slope was replaced by 2 : 1 slope (2 horizontal : 1 vertical). With M.W.L. both upstream and downstream this gave a well-shaped standing wave and the scour was not deep; but when the tail water was low the standing wave disappeared and the jet got bodily deflected with hypercritical velocity. This of course caused very deep scours. Thus sloping sills flatter than 1 : 1 were found unsuitable for the situation. *Vide* Figure 4 C-11(D).

Experiments 9 and 10 were carried out with a stilling basin 2½ feet deep, the other dimensions being the same as before and the end sill being vertical. For normal conditions of upstream and downstream water levels this proved all right, but with the downstream water very much lower than normal, deflection of the jet took place and the design had to be rejected. *Vide* Figure 4 C-11(E).

Experiments 11 and 12.—The above studies indicated that a good design might be obtained by improving the design tested in Experiments 5 and 6 by converting the sloping sill into a stepped sill. The 5 feet deep basin was therefor provided with.

CONCLUSION

The original design was defective in that no proper standing wave was formed for high afflux and in spite of long aprons deep scours were produced in the bed below.

The revised design as finally evolved was satisfactory. For all conditions of flow with high afflux a standing wave would form in the basin and the scour for the worst case was negligible.

The revised design being superior would cost very little towards maintenance. Even in initial cost a saving was effected by eliminating the second and third aprons. This saving amounted to Rs. 20,000.

(5) STILLING BASIN FOR MALAMPUZHA OVERFLOW DAM (7)

ABSTRACT

A series of experiments were carried out for obtaining an efficient design for a stilling basin for the Malampuzha overflow dam. This was a typical case of a dam with a small overflow depth and the studies are reproduced in this note.

Hydraulic particulars.—The design of the overflow dam experimented on is given in Figure 4 C.10. The relevant hydraulic particulars are furnished below :—

River bed level (average)	+ 278·0
Crest of dam	+ 362·5
M.W.L. upstream	+ 370·0
M.W.L. downstream	+ 290·0
Discharge per foot run	78 c/s.

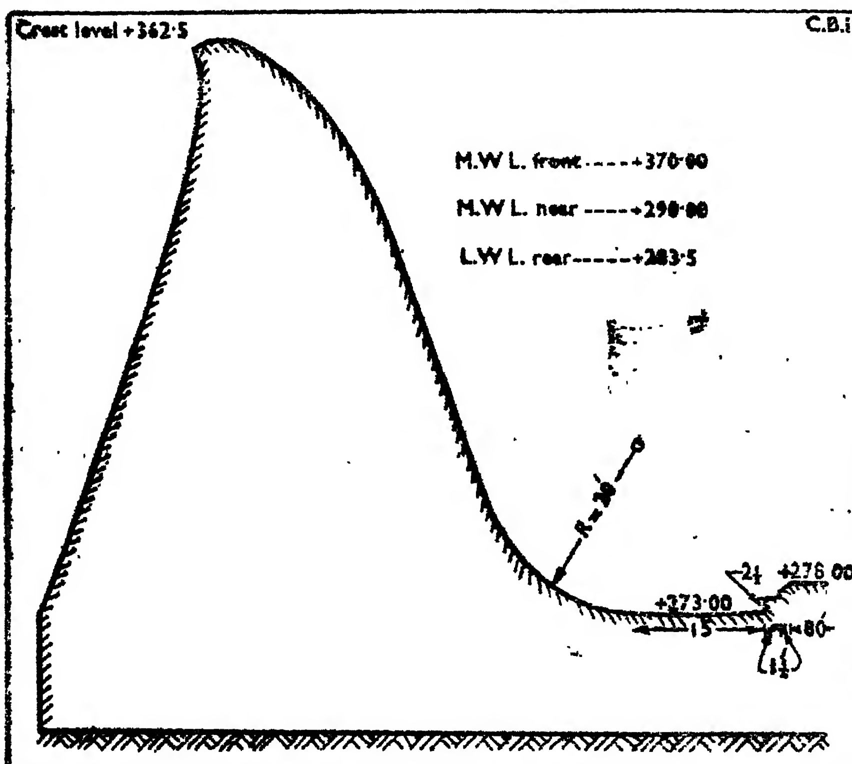

Figure 4 C.10 : *Showing the cross-section of Malampuzha over flow dam model.*

(7) Irrigation Research Station, Madras, Annual Report, 1947 pages 58-64.

MODEL

The model was a two dimensional one, made in teakwood to scale 1/100 and fitted up in the Laboratory Hydraulic Flume 40 feet × 2 feet × 2 feet. Downstream of the model seived sand of 24-30 mesh per inch grade was packed to a depth of ¾ foot. All experiments were made for two conditions, namely, (1) with maximum water level upstream and maximum water level downstream and (2) with maximum water level upstream and a low water level (plus 283·5) doswnstream. Water supply was drawn from the overhead tank, measured over the precalibrated weir in the flume and stilled properly. The downstream water levels were adjusted by a tail gate. The runs were made for two hours in each case, which period was found sufficient to give a stabilized bed downstream. All measurements of water surface profiles and bed scours were made by pointer gauges reading to a thousandth of a foot.

Experiments.—The following eight designs were tested on the model. With the experience we had gained from the several studies carried out on other spillway models it was possible to choose and select a few designs for test and avoid the necessity of going through a very large number of alternatives. In these experiments the main intention was to obtain a stilling basin which would ensure the formation of good standing waves for a wide range of discharge and water-level conditions.

Experiments 1 and 2 were made without a sill at the end of the basin. The standing wave profiles and scours obtained are shown in Figure 4 C.11(A). These two experiments had no practical significance and were merely intended to see the nature and extent of scour that might develop if no protective work were provided.

A sloped sill having two steps each 2½ feet rise and 2½ feet tread *vide* Figure 4 C.11 (F), was a clear improvement over the previous designs. Better standing waves were formed for all conditions of flow and scours were less deep.

Experiments 13 *and* 14.—A further improvement was effected on the above by cutting off the upper half of the top step at 45 deg. as shown in Figure 4 C.11(G). The standing waves were better and generally satisfactory and it was thought unnecessary to try for further improvement by providing friction blocks inside the basin or by other means.

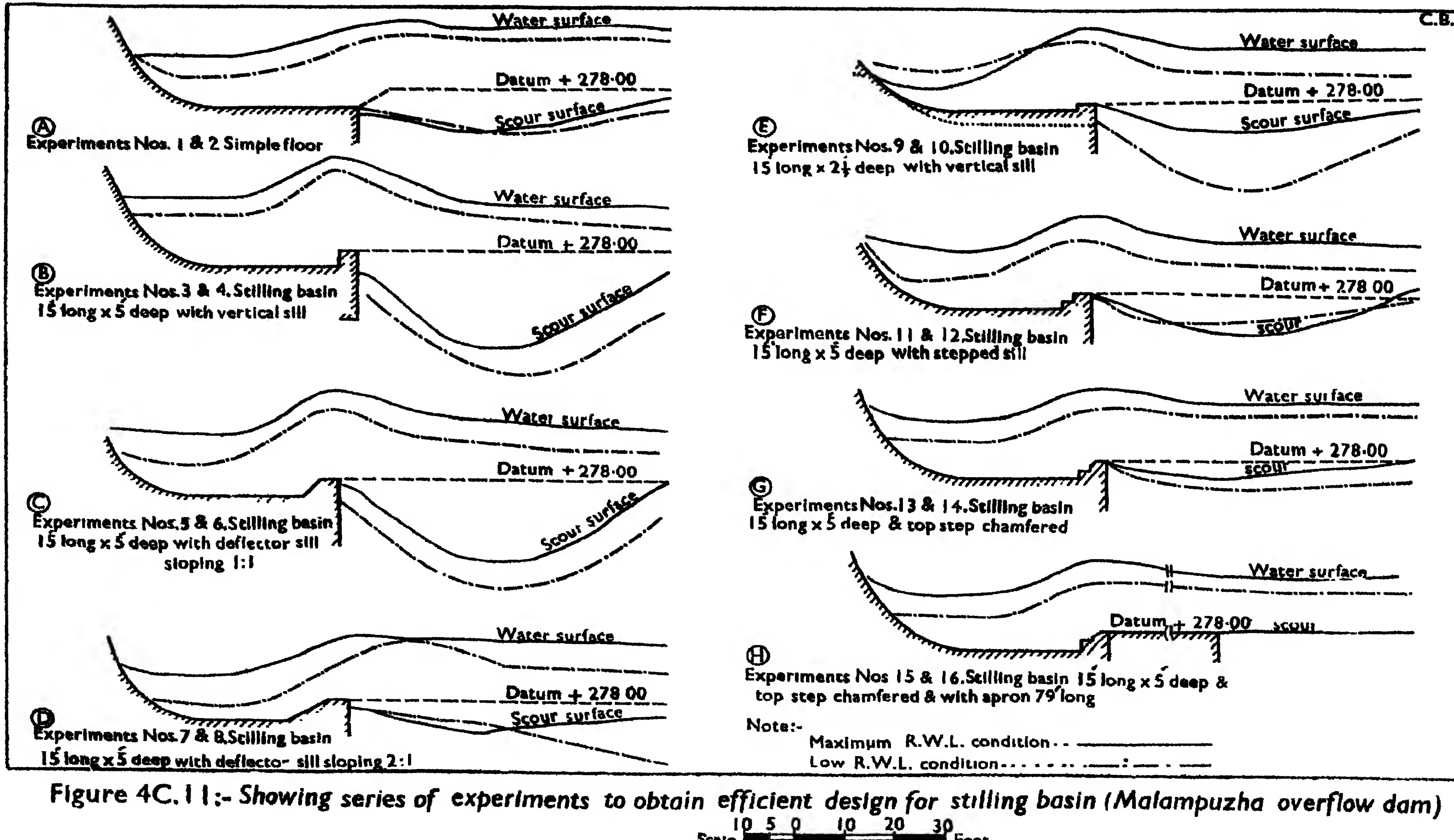

Figure 4C.11:- *Showing series of experiments to obtain efficient design for stilling basin (Malampuzha overflow dam)*

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TABLE 4 C.2

Statement showing Abstract of Results obtained in the Experiments on the ***Malampuzha Overflow Dam Model***

Experiment number	Description of the basin sill, *etc.*	Water levels		Whether a S.W. was formed on the jet got depleted	Level of the crest of S.W.	Distance of the crest from the toe	Deepest scour	Distance of the deepest scour beyond the toe	Remarks
		Upstream	Downstream						
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	Length of the basin 22'—6" with no end sill.	+370	+290	Good S. W. was formed.	+293·5	6·4	+267·3	20 feet	Not of practical importance.
2	Same as above ..	+370	+283·5	Do.	+290	6·4	+266·4	30 feet	Do.
3	Clear length of basin 18' and a rectangular sill 5' deep and 4½' wide at the end.	+370	+290	The S.W. rose unduly high and dived down giving rise to secondary waves	+300·8	The S.W. was formed within the cistern at a distance of 1·5' in U.S. of the sill.	+254·0	27 feet from the tip of the sill.	Depth of cistern appeared to be too much.
4	Same as above ..	+370	+283·5	Same as above ..	+297·5	Do.	+248·9	37 feet from the tip of the sill.	Do.
5	With a deflector having a slope of 1 : 1 added U.S. of the vertical sill.	+370	+290	S.W. was formed ..	+300	At the U.S. tip of the baffle.	+258·7	Do.	Fairly good design.
6	Same as above ..	+370	+283·5	Do.	+295·5	Do.	+252·3	..	Do.
7	With a deflector having a slope of 2 horizontal : 1 vertical sill.	+370	+290	Good S.W. was formed.	+292	At the D.S. tip of the baffle.	+270·4	27 feet from the tip of the sill.	Good for normal conditions, but unsatisfactory with low tail water.

TABLE 4-G.2—*contd.*

Experiment number	Description of the basin sill, etc.	Water levels		Whether a S.W. was formed on the jet got depleted.	Level of the crest of S.W.	Distance of the crest from the toe	Deepest scour	Distance of the deepest scour beyond the toe	Remarks
		Upstream	Down-stream						
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
8	Same as above ..	+370	+283·5	No S.W. was formed The jet got deflected bodily.	..	..	..	The scour was unsymmetrical. The level of the deepest scour at centre is +263·5.	Good for normal conditions but unsatisfactory with low tail water
9	Depth of the basin was reduced to 2½'. Thus a sill 4½' wide and 2½' deep was had at end of basin.	+ 370	+290	Standing wave was formed but had its crest rising above tail water.	+296·5	At a distance of 3 feet from the U.S. tip of the baffle.	+271·3	At a distance of 27 feet from the tip of the sill.	Better than all the previous designs but the danger of deflection of the Jet was present.
10	Do.	+370	+283·5	The S.W. dived down more prominently.	+293·2	At a distance of 0·5' from the U.S. tip of the baffle.	+256·6	Do.	Do.
11	Depth of the basin kept at 5' and with stepped sill at the end. Tow steps 2½' × 2½'.	+370	+290	A moderately good S.W. was formed.	+296·0	At a distance of 1 foot in front of U.S. tip of top step.	+267·7	At a distance of 27 feet from the tip of the sill.	Generally satisfactory
12	Same as above ..	+370	+283·5	Do.	+290·8	At a distance of 1·5 feet in the rear of U.S. tip of top step.	+270·2	Do.	Do.
13	The same design as above but with the top step chamferred	+370	+290	A very good standing wave was formed	+295·5	At the U.S. tip of the top step.	+273·5	At a distance of 21·2' from the toe of the sill.	Very satisfactory.
14	Same as above ..	+370	+283·5	Do.	+290·0	Do.	+270·5	At a distance of 27 feet from the tip of the sill.	Do.
15 & 16	Design as above and with an 80 feet apron added downstream.	..	..	Standing waves as above—Scours—*Nil.*					

Experiments 15 *and* 16.—These were repetitions of experiments 13 and 14 with the addition of an apron 80 feet long downstream of the stilling basin. In the prototype this can be obtained by levelling the rocky bed by chipping out projections or grouting up the cavities as the case may be. With this apron in position no scour was formed on the bed in rear, Figure 4 C.11(H).

CONCLUSION

Salient particulars of the observations made in the several experiments are tabulated in enclosed Table 4C.2. A stilling basin 15 feet long × 5 feet deep having a bucket of 20 feet radius on it upstream and a stepped sill of the dimensions shown in Figure 4C.11(H) at its end followed by an apron 80 feet long was found to be satisfactory. This gave a fairly efficient standing wave for all except very unusual conditions of flow and scour downstream was practically eliminated. A vertical sill induced a high standing wave accompanied by a dive and secondary waves. A sloping sill or a shallower cistern resulted in the bodily deflection of the shooting jet for certain discharge conditions.

(6) SPILLWAY DAM OF MANAIR IRRIGATION PROJECT (8)

ABSTRACT

Gives the results of experiments carried out on models of Manair Dam regarding the variation of the coefficient of discharge with different profiles and heights of the dam.

This dam is being constructed on the Manair and Kudlair Rivers just above their confluence in Medak district of the State. The main dam is a composite dam, *i.e.*, masonry facing with earth backing. In order to dispose off the total estimated flood discharge of 175,000 cusecs from the combined catchment of both the rivers, two separate spillway sections are being provided, one in the Manair gorge 1,075 feet in length and the other in the Kudlair gorge 1,639 feet in length. In addition a detached F. O.F. Weir 1,200 feet in length is being constructed. The following are the details in connection with the reservoir :—

(8) Hyderabad Engineering Research Laboratories, Annual Report, 1947, pages 8—11.

Maximum flood discharge by Ali Nawaz Jung Formula, 175,000 cusecs to be disposed off as follows with a head of 6 feet.

Kudlair and Manair spillways, 138,500 cusecs.

F. O. F. Weir .. .. 36,500 cusecs.

The sanctioned profile of the spillway section which is under construction is given in Figure 4 C.12. While the foundations for the spillway dam in the Kudlair gorge were being excavated, it was found that for a length of nearly 250 feet a horizontal fissure with a *morrum* seam occurred in the sheet rock foundation, and as it was considered inadvisable to construct the spillway dam on such a poor foundation, it was decided to alter the profile to give a higher coefficient so that the length of the spillway dam may be reduced.

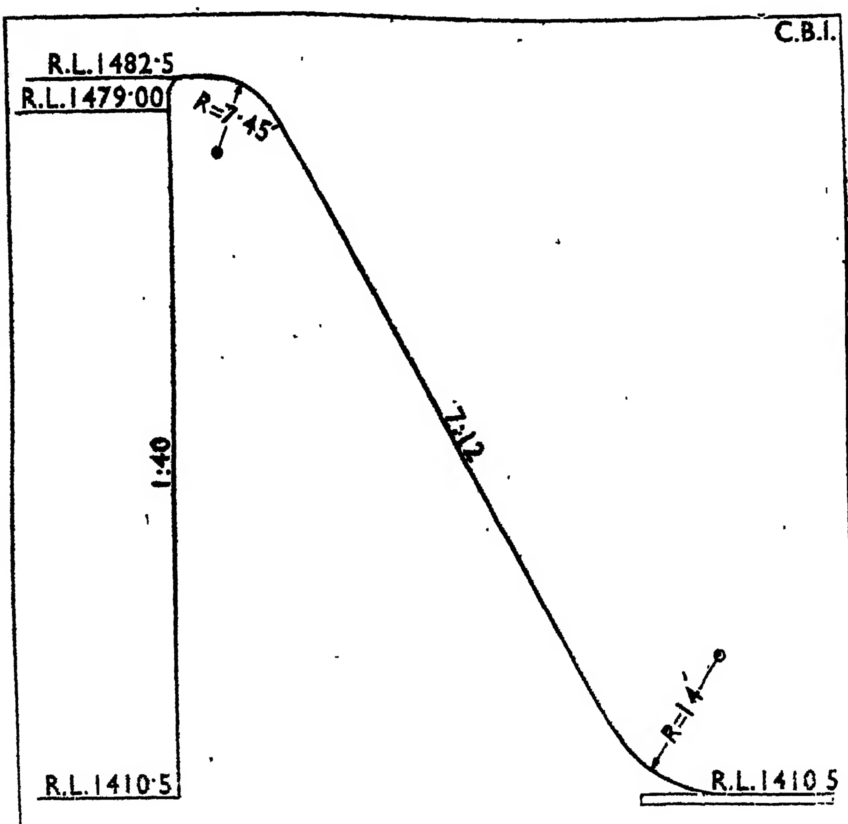


Figure 4 C.12 : *Showing profile of Manair spillway as approved for construction.*

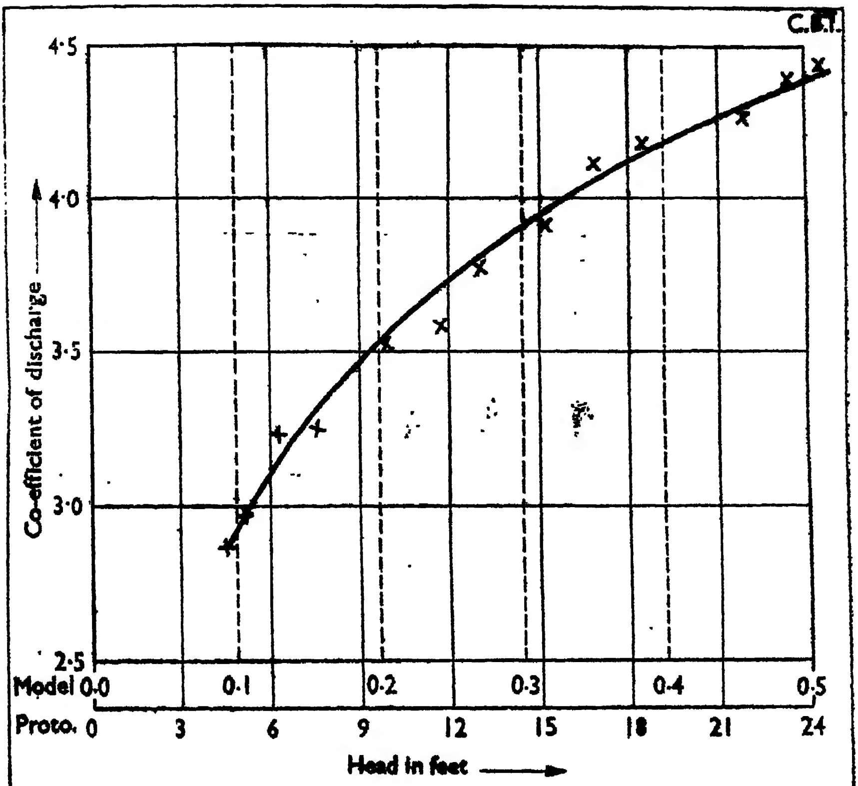


Figure 4 C.13 : *Showing coefficient of discharge for Manair spillway. (Section of Figure 4C.12, 1/48 scale model).*

PROBLEM

This reduction of 250 feet in length of the spillway dam requires a coefficient of 4·00 as against the assumed coefficient of 3·5 of the sanctioned section. As part of the spillway section is already constructed, only the top 10 feet to 15 feet could be altered to give the higher coefficient needed.

EXPERIMENTS AND RESULTS

At first a model of the sanctioned profile was constructed in flume No. 3, two feet wide, with glass sides. The scale of the model was 1/48. On this scale this model gave a coefficient of 3·1 at the required prototype head of six feet. Figure 4 C.13 gives the coefficients obtained for this section. Pressures were also taken and these are given in Figure 4 C.14. Table 4 C.3 gives the results obtained with this profile.

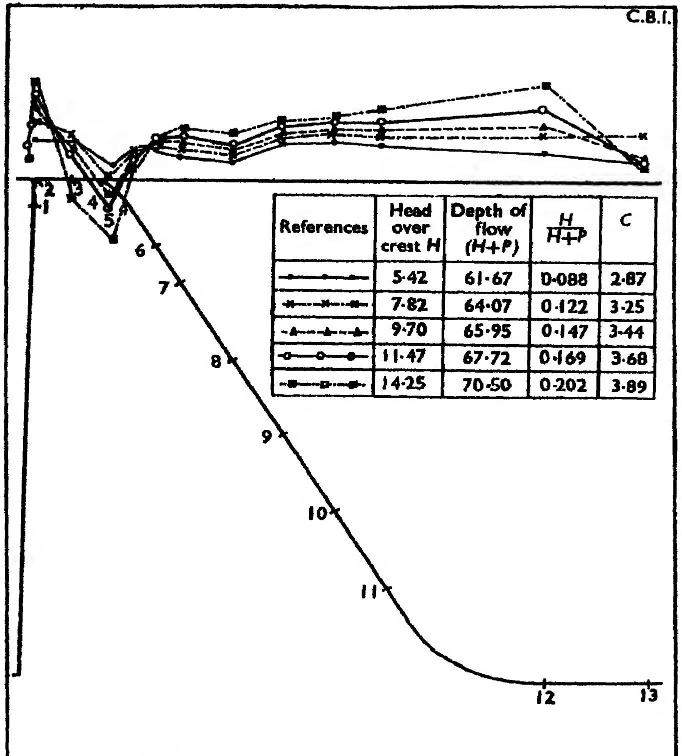
TABLE 4 C.3

Pressures observed at various points on the sanctioned spillway section of Manair Project, Scale $\frac{1}{48}$.

Pipe No.	Depth of pipe below crest in feet	Pressure head in feet of water ; Atm. pr. being taken as datum.				
		$H=5\cdot42$	$H=7\cdot42$	$H=9\cdot70$	$H=11\cdot47$	$H=14\cdot25$
1 ..	3·00	3·2	4·0	4·2	3·6	2·0
2 ..	0·50	4·4	6·6	8·0	9·4	11·0
3 ..	0·00	4·4	4·8	3·6	2·4	—2·6
4 ..	0·50	1·6	0·0	—2·0	—3·6	—7·2
5 ..	3·00	3·2	3·2	2·8	2·4	1·2
6 ..	7·50	3·2	3·8	4·2	4·2	4·0
7 ..	11·75	2·8	3·6	4·2	4·8	5·6
8 ..	20·50	2·0	2·6	3·4	3·8	5·2
9 ..	28·00	4·4	4·8	5·2	6·0	6·6
10 ..	37·50	4·4	5·2	5·6	6·2	7·0
11 ..	46·00	4·0	4·8	5·6	6·2	8·0
12 ..	57·25	3·2	5·6	6·0	8·0	11·2
13 ..	57·25	2·0	5·2	2·0	1·2	1·2

Next the profile was altered substituting the upstream chamfer with a circular arc and the downstream circular arc with a parabolic curve. After a few trials, the profile given in Figure 4 C.15 was obtained which gave negative pressures at only a very few points and that too of very slight intensity at the required head of 6 feet. This has an upstream circular curve of radius 225 feet and the downstream parabola whose equation is $Y = \frac{(4x)^2}{15}$ in such a way that it is tangential to the downstream slope of the sanctioned profile at a depth of ten feet below the crest.

Tests carried out on 1/48 scale model of the section so obtained gave a coefficient of 3·7. The same profile was next constructed in the two feet channel outside the Laboratory to a scale of 1/36. The results obtained on 1/48 and 1/36 scale models of the new profile are given in Figure 4 C.16 and Figure 4 C.17. The pressures obtained for the 1/48 scale model of this profile are given in Figure 4 C.18 and Table 4 C.4 gives the results of the observations :—



References	Head over crest H	Depth of flow (H+P)	$\frac{H}{H+P}$	C
—·—·—	5·42	61·67	0·088	2·87
—x—x—	7·82	64·07	0·122	3·25
-▲-▲-	9·70	65·95	0·147	3·44
—o—o—	11·47	67·72	0·169	3·68
—■—■—	14·25	70·50	0·202	3·89

Figure 4C.14:- *Showing pressure curves at various heads for Manair Spillway profile of figure 4C.12*

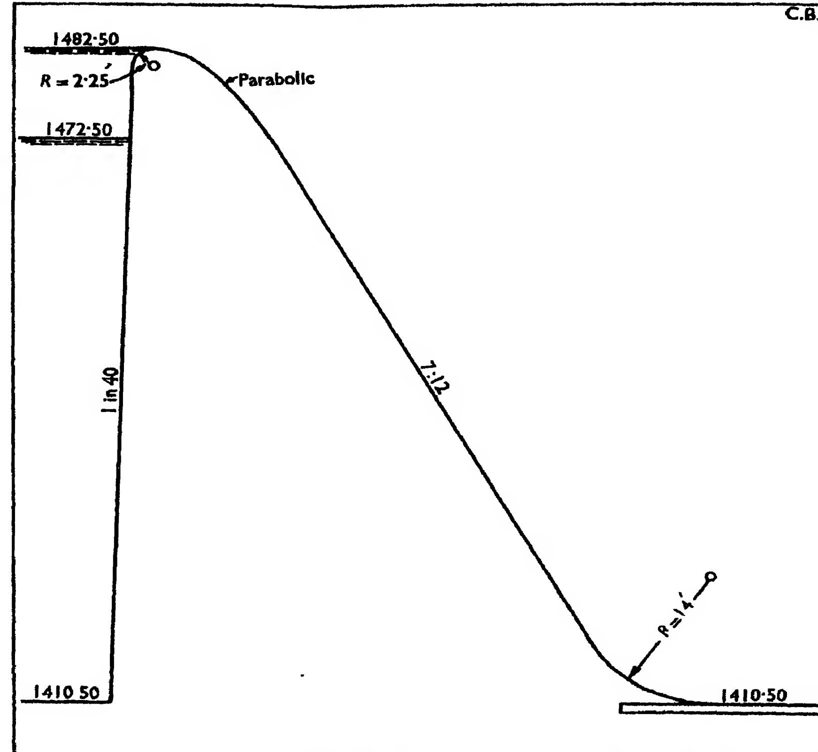


Figure 4C.15:- *Showing profile recommended for Manair Spillway*

Scale 10 5 0 10 20 30 feet

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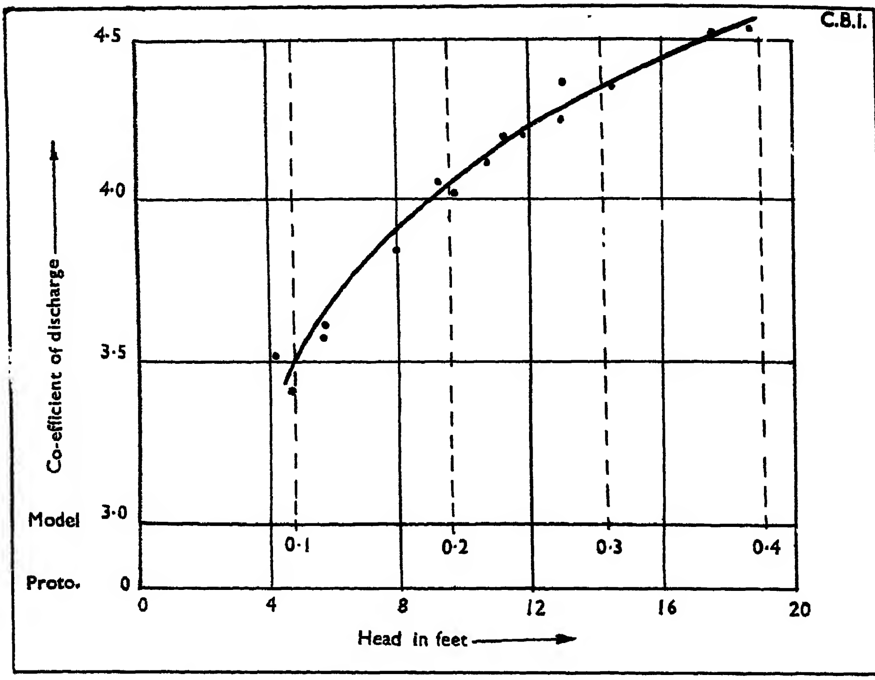

Figure 4C.16:- *Co-efficient of discharge for the recommended Manair Spillway section for $\frac{1}{48}$ scale model*

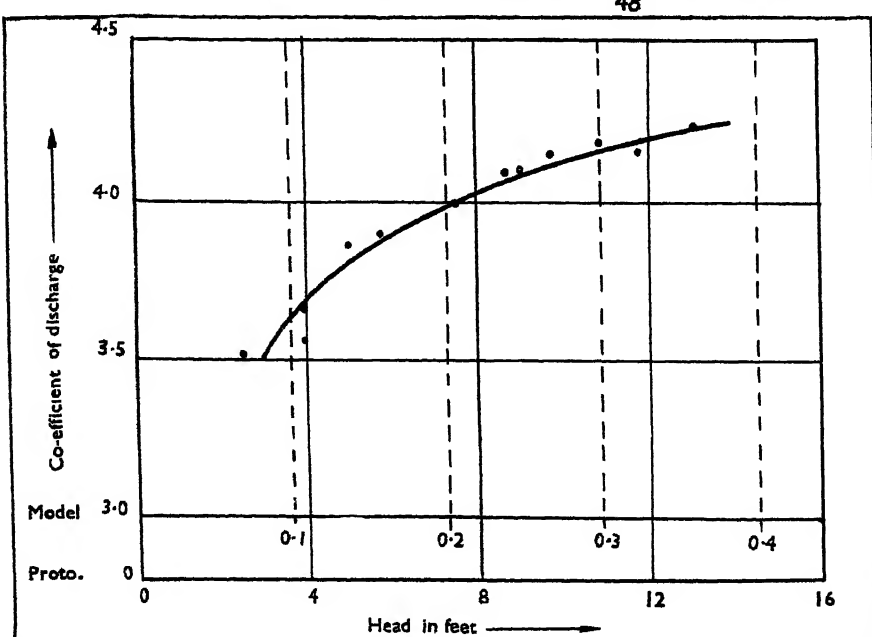

Figure 4C.17:- *Co-efficient of discharge for the recommended Manair Spillway section for $\frac{1}{36}$ scale model*

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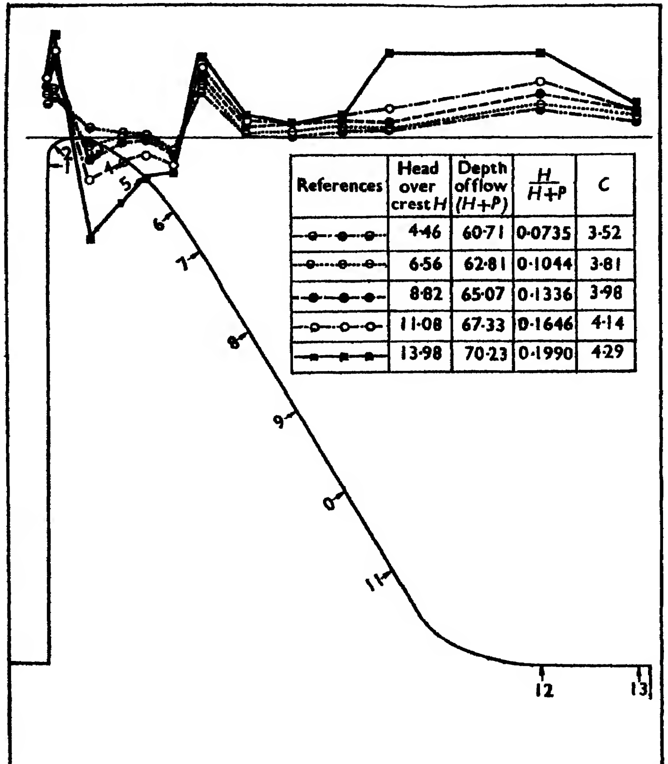

Figure 4C.18:- *Pressure curves at various heads for the recommended Manair Spillway section for $\frac{1}{48}$ scale model*

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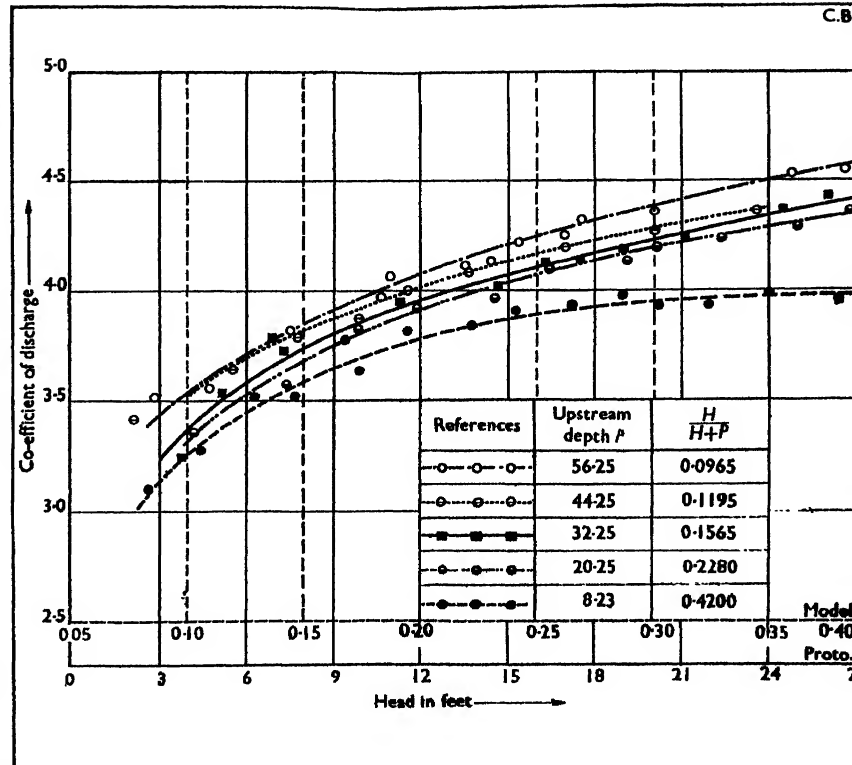
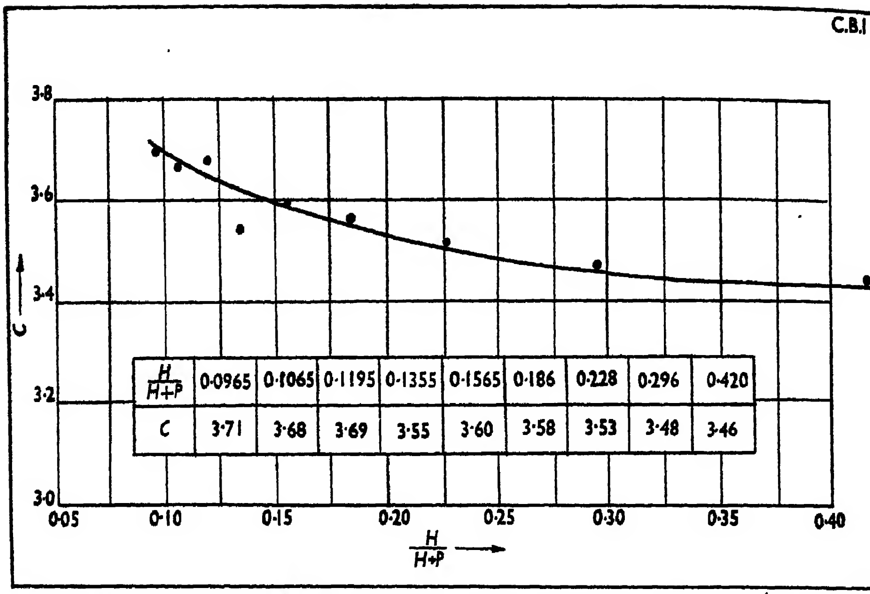
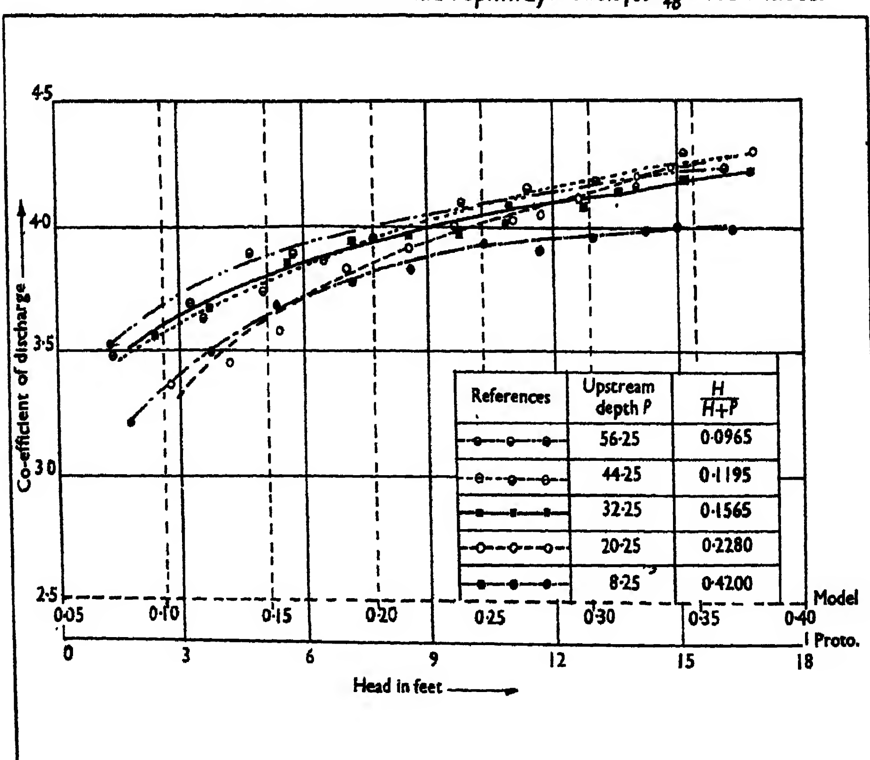


Figure 4C.19:- *Coefficient of discharge for various values of P for the recommended Manair Spillway section for $\frac{1}{48}$ scale model*



$\frac{H}{H+P}$	0·0965	0·1065	0·1195	0·1355	0·1565	0·186	0·228	0·296	0·420
C	3·71	3·68	3·69	3·55	3·60	3·58	3·53	3·48	3·46

Figure 4C.20:- *Curve between C and $\frac{H}{H+P}$ at the required proto head of 6′ for the recommended Manair Spillway section for $\frac{1}{48}$ scale model*



References	Upstream depth P	$\frac{H}{H+P}$
	56·25	0·0965
	44·25	0·1195
	32·25	0·1565
	20·25	0·2280
	8·25	0·4200

Figure 4C.21:- *Coefficient of discharge for various values of P for the recommended Manair Spillway section for $\frac{1}{36}$ scale model*

Reg. No. 2737 XDD (C) '50-1,210 (P Z O.).

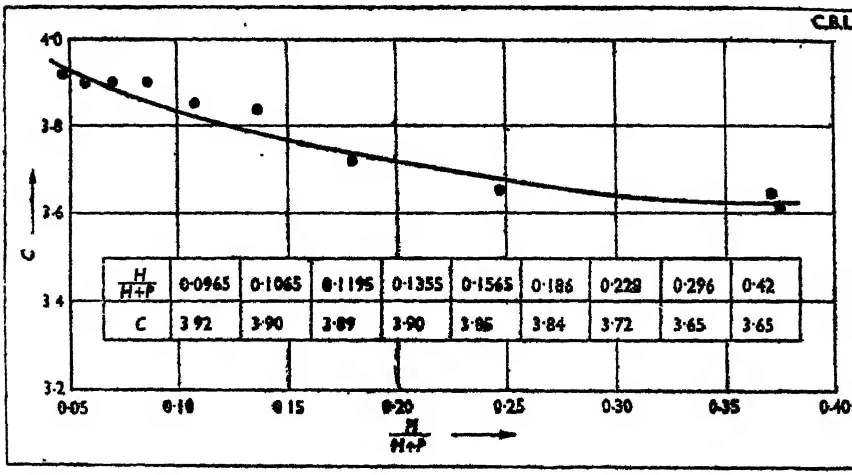
TABLE 4 C.4

Pressures observed at various points on the final profile of Manair Spillway section.

Pipe No.	Depth of pipe below crest in feet.	Pressure head in feet of water : Atm. Pr. being taken as datum.				
		H=4·46	H=6·56	H=8·82	H=11·08	H=13·98
1 ..	3·00	3·2	4·8	5·6	6·4	6·4
2 ..	0·75	4·0	5·6	8·0	9·6	11·6
3 ..	0·25	0·8	—0·8	—2·8	—4·8	—11·6
4 ..	1·75	0·2	0·0	—1·2	—2·8	—7·2
5 ..	4·00	0·2	0·0	—0·8	—2·0	—4·4
6 ..	7·75	—1·6	—2·0	—2·4	—2·8	—4·0
7 ..	11·75	4·8	6·0	6·4	7·2	8·0
8 ..	20·50	0·0	0·4	0·6	0·8	1·2
9 ..	29·00	0·0	0·4	0·6	0·6	0·8
10 ..	37·50	0·4	0·6	0·8	1·0	1·6
11 ..	46·00	0·8	0·8	0·6	3·2	9·2
12 ..	57·25	2·8	3·6	4·4	6·0	8·8
13 ..	57·25	1·6	2·0	2·4	2·4	2·8

As the upstream bed level varies for different portions of the spillway it was proposed to obtain the value of the coefficient for different values of $\frac{H}{H+P}$ where H is the head over the crest of weir and P is the upstream depth below crest of the weir. So experiments were conducted on the 1/48 scale model of this profile, for different values of P. Figure 4 C.19 gives the coefficient obtained for various values of P. Figure 4 C.20 gives the curve between C and $\frac{H}{H+P}$ for the required prototype head of six feet.

Similarly Figure 4 C.21 gives the coefficient for various values of P and Figure 4 C.22 the relation between C and $\frac{H}{H+P}$ for the prototype head of six feet on the 1/36 scale model of the profile. Similar experiments will be conducted on the 1/24 scale model and it is proposed to repeat the same on 1/12 scale model of the same section to enable the coefficients to be extrapolated.



$\frac{H}{H+P}$	0·0965	0·1065	0·1195	0·1355	0·1565	0·186	0·228	0·296	0·42
C	3·92	3·90	3·89	3·90	3·86	3·84	3·72	3·65	3·65

Figure 4 C.22 : *Showing curve between C and $\frac{H}{H+P}$ at the required Proto head of six feet for the recommended Manair spillway section for 1/36 scale model.*

From these it appears that the ratio $\frac{H}{H+P}$ affects the coefficient slightly, up to about 6 per cent. in the case of over flow dams of this type.

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(7) HYDRAULIC DESIGN OF THE NANGAL BARRAGE(*)

ABSTRACT

The Central Technical Power Board recommended that, in order to utilise all the energy available from Bhakra and Nangal Projects, the maximum winter pond level in the Nangal pond might be raised from R. L. 1140.0 to R. L. 1154·0.

The hydraulic design of the Nangal Barrage was accordingly revised by the East Punjab Central Design Office, and the Institute asked to carry out model experiments for full information on some of the design features.

(*) East Punjab Irrigation Research Institute, Amritsar, Annual Report, 1947, pag 24-28.

In addition, the Institute revised some aspects of the design and sent a note to the Chief Engineers. A summary of the review is given.

DISCHARGE COEFFICIENT

The design assumed that the value of the coefficient C in the formula $Q = C\,B\,H^{3/2}$ would be equal to 3·10. It was pointed out that if the floor upstream of the crest silted to R.L. 1,112·0, which was the assumed R.L. of the silted bed upstream of the barrage, it would become flush with the crest, of which the R.L. was also 1,112. In such a case, the value of C might not agree with that assumed, though actual experiments alone could indicate the extent, if any, of the likely difference.

ENERGY CALCULATIONS FOR FINDING CISTERN LEVEL

The calculations for E_{f_2} for given values of q and H_L, which were later utilised for finding the R.L. of cistern floor, involved the estimation of the downstream depth, D_2, as a first step. As the values of q and H_L involved in the design lay beyond the range of the curves given in plate XI. 2 of the Central Board of Irrigation Publication No. 12 'Design of weirs on permeable foundations' the estimation of D_2 was probably done otherwise by trial and error, but no details were given.

It was pointed out that the empirical formula ($D_2 = q^{1/2}\,H_L{}^{1/6}$) given by Dr. Malhotra in the January 1947 issue of the C.B.I. Journal could have been used for finding the value of D_2 to a first approximation. As it was, it gave values very closely agreeing with those derived by the Central Designs Office, as shown in Table 4C.5.

TABLE 4C.5

Q	H_L	D_2 (estimated by C.D.O)	$D_2 = q^{1/2}\,H_L{}^{1/6}$
14·3	51·34	7·29	7·30
28·6	48·10	10·2	10·2
71·4	29·76	14·6	14·9
142·8	24·94	20·3	20·4
214·3	21·78	24·6	24·5
439·5	14·64	33·8	32·7

GLACIS PROFILE

The Central Designs Office considered only the following two alternatives as suitable profiles for the barrage glacis :—

(*i*) Simple trajectory with the maximum pond level and a small gate opening.

(*ii*) Montagu's so-called ' semi-gravity ' parabola.

For (*i*) the crest velocity was estimated as follows :—

Maximum R.L.=1,154·0. Crest R.L.=1,112·0.

Gate opening 1·0 foot. R.L. of stream at half depth, when leaving the crest= 1,112·5.

Head=1,154=1,112·5=41·5

$$\text{Velocity at crest} = C\sqrt{2gh.}$$
$$=0\cdot98.\sqrt{2 \times 32\cdot2 \times 41\cdot5}$$
$$=50\cdot7 \text{ feet per second.}$$

For (*ii*) the crest velocity was taken as equal to the velocity obtaining with free flow over the crest with the maximum possible discharge intensity (438 cusecs per foot run) *i.e.* velocity at crest$=(g.\,q)^{\frac{1}{3}}=(438 \times 32\cdot2)^{\frac{1}{3}}=23\cdot8$ feet per second.

The equation for (*i*) was taken as that of the gravity parabola, $y=g\,x^2/2\,U^2$ with $U=50\cdot7$ and that for (*ii*) as that given by Montagu *viz.*

$$x=y+\sqrt{4U^2\,y/g}, \quad \text{with } U=23\cdot8.$$

The two curves being not very different, the Central Designs Office chose the simple trajectory in preference to the Montagu type, as the former obviated the possibility of cavitation.

It was pointed out by the Institute that the Montagu formula was based on a fallacy, as shown by the Lahore Institute some years earlier. Also that the most economical glacis profile (*i.e.* which gave the maximum horizontal acceleration to the jet leaving the crest, in a given horizontal length of the work, for known values of h and U) was given by the formula due to Dr. J. K. Malhotra, *viz.*

$$x=y\sqrt{1+\frac{U^2}{2gh}}+\sqrt{\frac{2U^2y}{g}} \qquad (40.8)$$

where h was the fall between the crest and the toe of the glacies, and U the crest velocity.

Taking $U=23 \cdot 8$, as in the Montagu design, the Equation (4C.8) was reduced

to $x=1 \cdot 18\, y+5 \cdot 95 \sqrt{y}$ (4C.9)

The values of x for different values of y are given in Table 4 C.6.

TABLE 4 C.6

Elevation (R.L.)	y	x
1,112·0	0	0
1,110·0	2	10·76
1,107·0	5	19·18
1,102·0	10	30·59
1,097·0	15	40·70
1,092·0	20	50·15
1,089·0	23	55·60

The glacis given by these values was more economical than the trajectory and Montagu designs, Figure 4 C.23, the overall length being 91 per cent. of the trajectory and 88 per cent. of the Montagu designs. But as the trajectory design appeared more likely to avoid cavitation for small gate openings, when the crest velocity would be higher than that obtaining in free-flow, it was concluded that the choice of the trajectory design was justified.

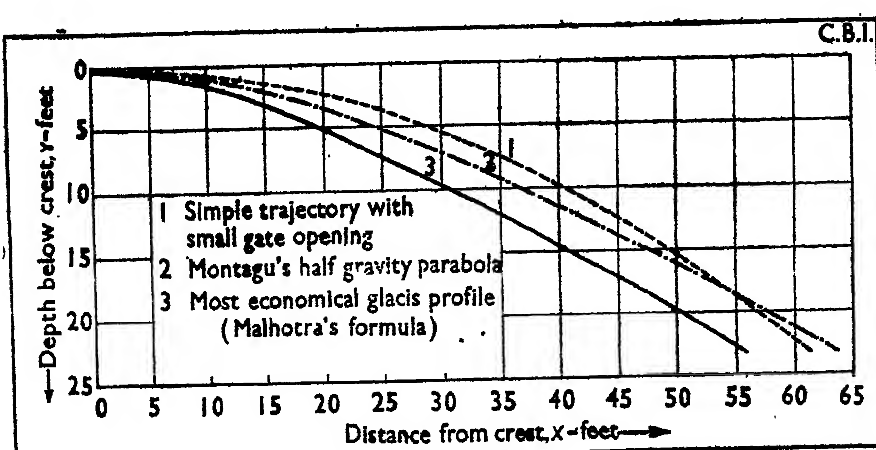

Figure 4 C.23 *Showing different designs of glacis profile (Revised hydraulic design of Nangal Barrage).*

It was also pointed out that the reverse curve at the toe of the glacis, while useful otherwise, was given a rather sharper curvature than a smooth transition required.

(*i*) The design assumed that the use of an upstream grout curtain and the foundation of the weir concrete right over the conglomerate would reduce the uplift pressures throughout the floor length by 50 per cent.

It was suggested that the assumption might be verified otherwise, for, if it were not fulfilled, the absolute values of the uplift pressures would go up substantially and the safety factor materially reduced.

(*ii*) It was also pointed out that the calculations for the safety factor could be simplified by using the following formula due to Dr. Malhotra, (assuming floatation gradient equal to $1 \cdot 0$) :—

$$\text{S.F.} = \frac{\sqrt{5d\ (b+d)}}{H} \quad \ldots \quad \ldots \quad \ldots \quad \ldots \qquad (4C.10).$$

b, d and H being respectively the floor width, toe wall depth and the head on the floor.

For Nangal barrage, $d = 9$ feet, $b = 333$ feet and H was taken as 50 per cent. of the maximum likely head (53 feet).

$$\text{So S.F.} = \frac{\sqrt{45 \times 342}}{26 \cdot 5} = 4 \cdot 68,$$

which agreed with the value otherwise obtained.

(*iii*) Further, it was suggested that in calculating the uplift percentages, another formula, also due to Dr. Malhotra, could be used, which gave the percentage head cut off by a curtain at the toe or heel of the barrage as :

$$p = 90\ \sqrt{d/b} \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \qquad (4C.11).$$

For Nangal barrage, the heel curtain was 15 feet deep and the toe curtain was 16 feet deep.

$$\therefore p\ (\text{heel}) = 90\ \sqrt{15/333} = 19 \cdot 0, \text{ and}$$

$$p\ (\text{toe}) = 90\ \sqrt{16/333} = 19 \cdot 6$$

which agreed well with the values obtained by the Central Designs Office.

(*iv*) Finally, it was suggested that the depth of the toe wall might be increased by another five feet or so, as a margin of safety against scour and retrogression, for which the existing provision was barely sufficient with the assumed maximum discharge intensity.

(8) MODEL EXPERIMENTS ON NANGAL WEIR(10)

ABSTRACT

To test the suitability and soundness of the proposed design for the Nangal weir, some preliminary experiments were made on a small scale model in the flume at Lahore. A sectional model of the Nangal Barrage was constructed to a scale of $\frac{1}{45}$ to the design as shown in Figure 4 C.24. Tests were carried out for determining :—

(a) The coefficient of discharge.
(b) The depth of scour for various discharge intensities.
(c) Pressure distribution on floor.
(d) Regulation of double gates.

COEFFICIENT OF DISCHARGE

The coefficient of discharge was calculated from the following formula for free-fall conditions.

$$C=E/(H+h_a)^{\frac{3}{2}},$$

Where E=discharge per foot run,

H=Head above the crest,

h_a=Head due to velocity of approach.

C=Coefficient of discharge.

The head due to velocity of approach was determined from $V^2/2g$ using the observed value of V and also its calculated value, calculating it from Q_m/A where A is the cross-section area above the crest. The results of the above tests are given in Table 4 C.7.

TABLE 4 C.7

Discharge intensity	Value of C	
	Using observed value of V	Using calculated value of V
437·5	3·331	3·385
375·0	3·330	3·348

(10) East Punjab Irrigation Research Institute, Amritsar, Annual Report 1947, pages 29-32.

DEPTH OF SCOUR.

For scour tests the model was run for different discharge intensities both with concentration of flow and without concentration of flow. These test were carried out first with discharges of 50,000 cusecs to 200,000 cusecs assuming 20 per cent. concentration of flow and with discharges of 300,000 cusecs and 350,000 cusecs assuming 5 per cent. concentration of flow. In each case the model was run for three hours to obtain stable scour conditions. The results of the above tests and upstream and downstream water levels *etc.* are given in Table 4 C.8.

TABLE 4 C.8

Serial No.	Q	Upstream water level R.L.	Downstream water level R.L.	Scour depth below floor level in feet	Distance of max. scour from down-stream end of floor in ft.
1	437·5	1,136·0	1,126·0	35·0	100·0
2	375·0	1,137·0	1,124·6	32·0	80·0
3	327·4	1,138·0	1,123·0	29·0	70·0
4	285·7	1,139·0	1,121·0	26·0	60·0
5	214·3	1,140·0	1,118·8	19·0	50·0

Plottings of the scour depth, line of maximum velocity and water surface profile for the maximum discharge are shown in Figure 4 C-25 and Figure 4C.27

SCOUR-DEPTH TESTS WITH NO CONCENTRATION OF FLOW

Scour tests were also made with the following discharges assuming no concentration of flow *viz.*, 100,000 cusecs, 200,000 cusecs, 300,000 cusecs and 350,000 cusecs. In each case water surface profile and velocity observations were also made. The results of these tests are given in Table 4 C.9.

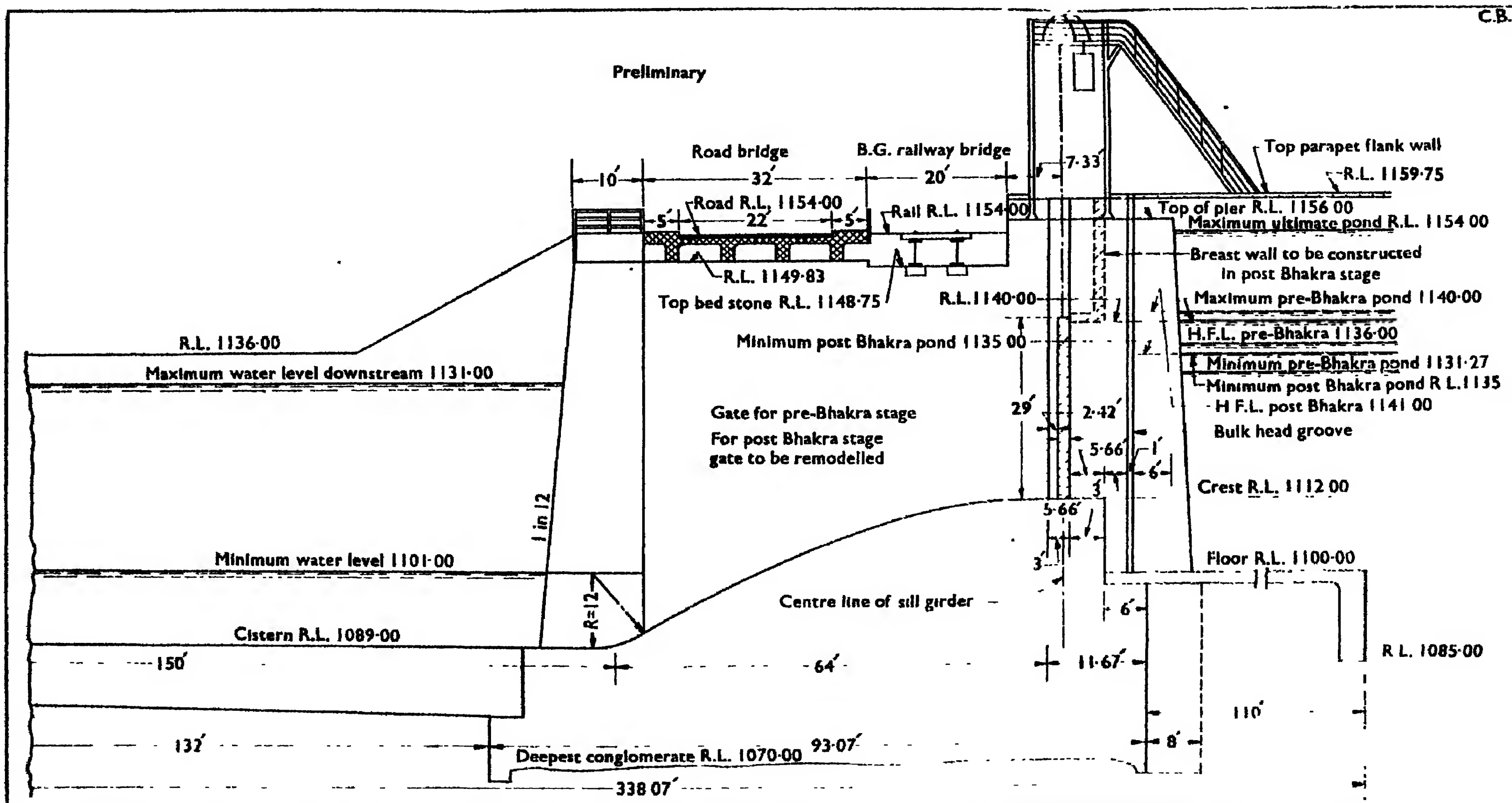

Figure 4C.24:- *Showing proposed maximum section with pond R.L. 1154·00 in post Bhakra stage (Nangal Barrage)*

Scale 25 20 15 10 5 0 25 50 75 Feet

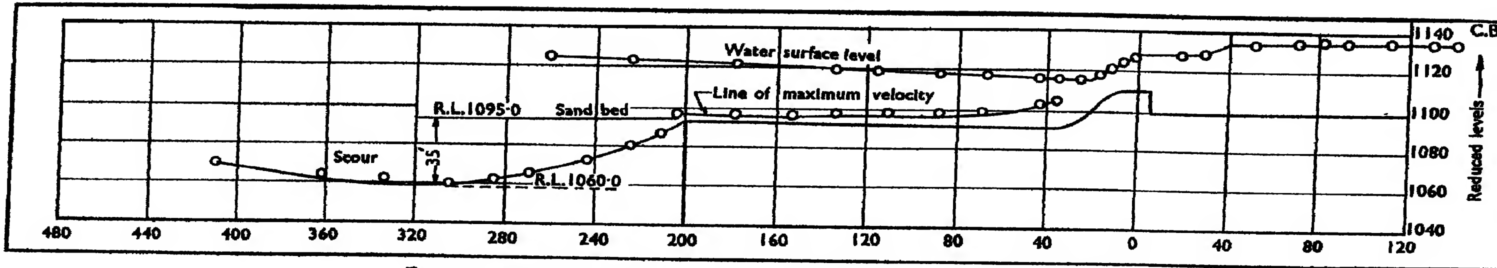

Figure 4C.25 :– *Showing Nangal Weir discharge 437·5 Cs. per ft. run*

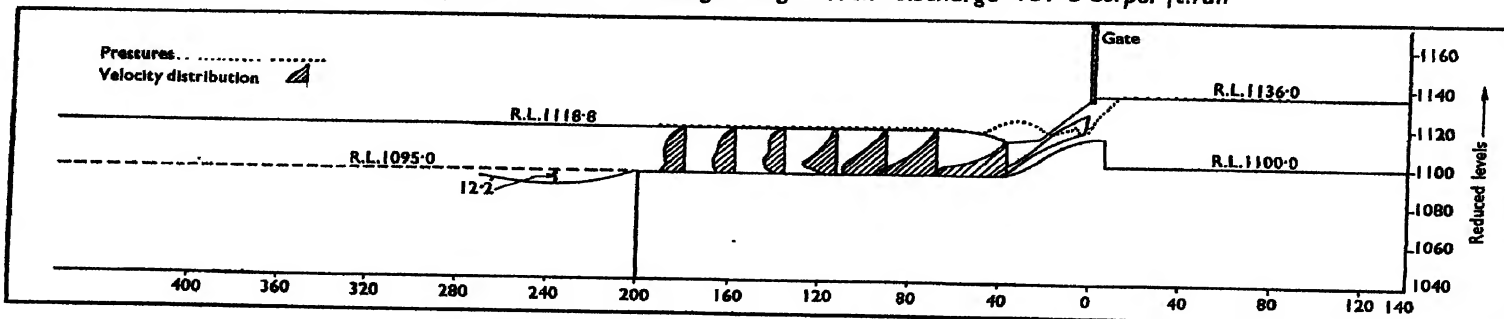

Figure 4C.26 :– *Showing Nangal Weir discharge 214·3 Cs. per ft. run*

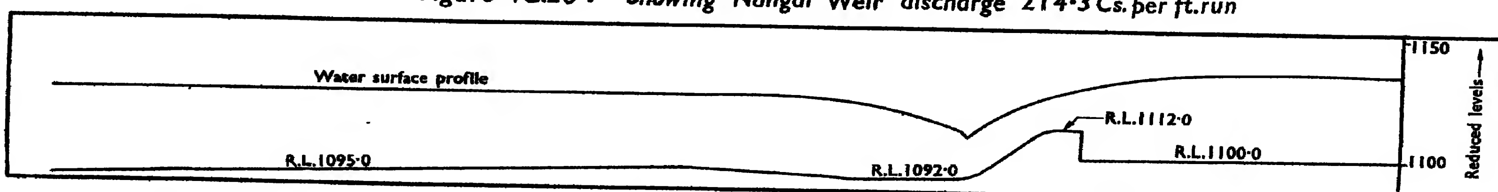

Figure 4C.27 :– *Section of Nangal Weir showing water surface profile in discharge equivalent to 437·5 Cs. per ft. run*

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TABLE 4 C.9

Discharge Q	Discharge intensity q	Upstream water level	Downstream water level	Scour depth in feet
100,000	119·1	1,140·04	1,115·0	4·6
200,000	238·1	1,139·0	1,121·0	16·0
300,000	357·1	1,137·0	1,124·6	25·0
350,000	416·7	1,136·0	1,126·0	29·3

DOUBLE GATE REGULATION AND PRESSURE PLOTTINGS

It was laid down in the design that as soon as silt is passing in the water from May onwards, the pond will be reduced to 1,136·0. Tests were, therefore, carried out with pond level 1,136·0 and discharge intensities 71·4, 142·9 and 214·3 cusecs. Double gates were so regulated that the two jets intersected near the toe of the glacis. The upper and lower openings and scour depths are given in Table 4 C.10.

TABLE 4 C.10

Discharge intensity	Lower opening in feet	Upper opening in feet	Downstream water level	Maximum depth of scour in feet
71·4	1·0	2·475	1,111·0	1·0
142·9	1·5	5·220	1,115·7	7·0
214·3	3·0	8·685	1,118·8	12·0

The velocity distribution, interaction of jets, scour depth, water surface profile and pressures on the floor for the maximum discharge run are shown in Figure 4 C.26. No definite conclusions could be drawn from these preliminary tests.

A large scale model of the Nangal weir, consisting of three bays, to a geometrical scale of 1/15 according to the original design was constructed at Malakpur. This is shown in Figure 4 C.27 (A).

Figure 4 C.27 (A): *A photograph showing a model of Nangal weir, consisting of 3 bays with staggered blocks arrangement.*

It was examined for the maximum discharge of 437·5 cusecs per foot run. Water surface profile as observed on the model is shown in Figure 4 C.27. The action downstream was heavy and scour to a depth equivalent to 34 feet occurred. The material forming the bed downstream of the *pacca* floor consisted of sand. Due to change in the pond level from 1,140 to 1,154 in the post Bhakra Stage, the design was changed to meet the new requirements, as shown in Figure 4 C.28. and the model was reconditioned accordingly. Investigations on this model were started on the following lines :—

(*a*) Protection downstream ;

(*b*) Design of cistern ;

(*c*) Design of glacis ;

(*d*) Coefficient of discharge ;

(*e*) Hydrodynamic pressures ,

(*f*) Cavitation.

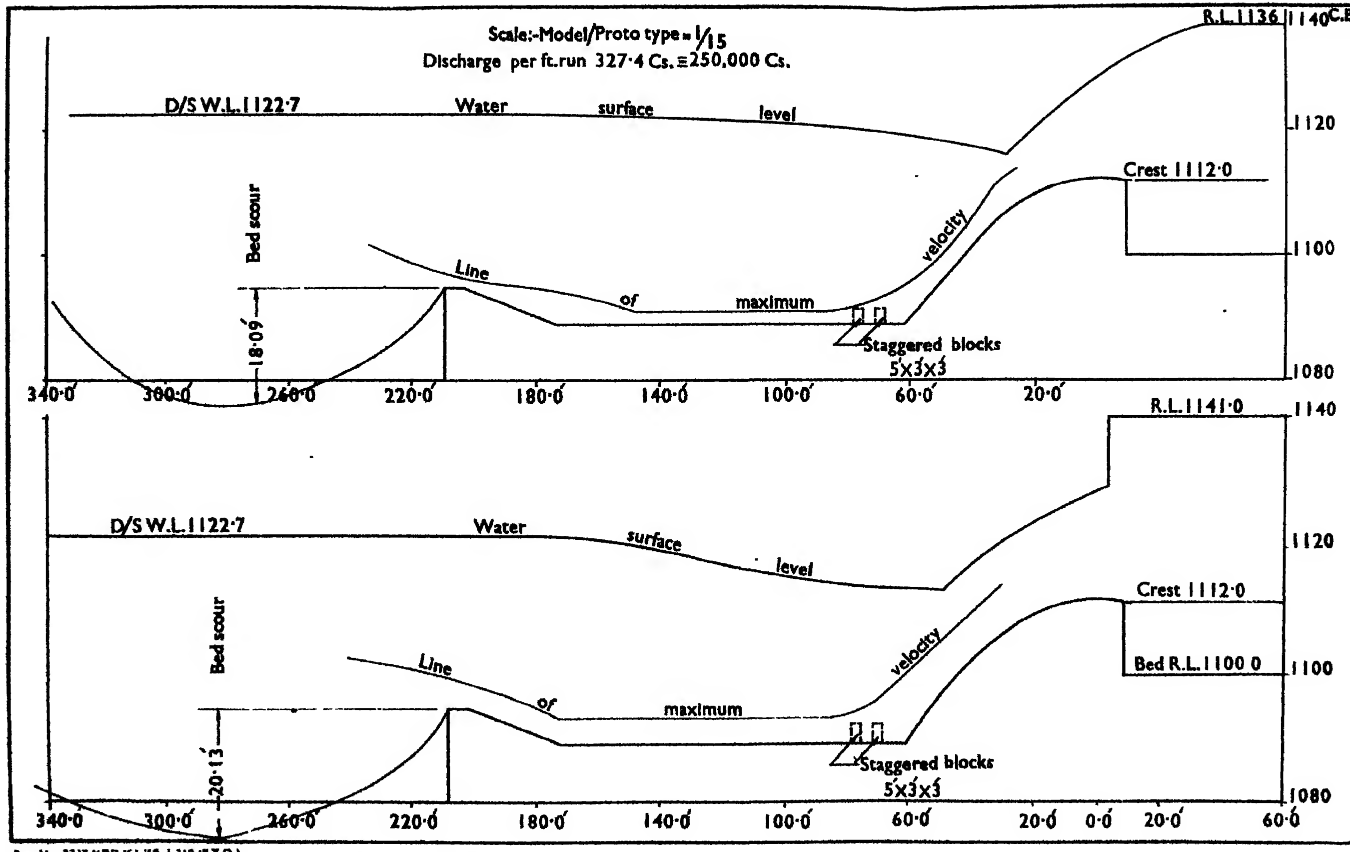

Figure 4C.28:- *Showing experiments on Nangal barrage model*

Up to the end of the year, the model was examined with the following discharges :—

Post Bhakra stage	Pre Bhakra stage
250,000 cusecs	250,000 cusecs
150,000 cusecs	150,000 cusecs
100,000 cusecs	100,000 cusecs

from plottings of the observations showing water surface profile, the lines of maximum velocity and the scour downstream it was seen that the line of maximum velocity remains near the bed in the cistern, it only temporarily shoots up over the slope and returns near the bed on the *kacha* bed causing considerable scour.

A view of the model of the weir operating with a discharge equivalent to 250,000 cusecs is shown in the photograph Figure 4 C.29. Without protection or with unsatisfactory protection action downstream is very heavy.

Figure 4 C.29 : ***A photograph showing the weir of the Nangal view of rating with a discharge equivalent to 250,000 cusecs.***

DISCUSSION BY THE RESEARCH COMMITTEE

MR. GUPTA introduced item (1) and said that downstream protection works for Nagwa Dam proposed last year *viz.*, a 90 feet long and 40 feet deep cistern could not be constructed at site as it was not possible to hewn the rock below bed level. As an alternative, a three feet high subsidiary weir at the end of 90 feet long floor at bed level and two feet high baffle at a distance of 40 feet from the toe of the main weir was found equally effective.

A geometrically similar model of the whole weir and allied works was constructed to scale 1:36 to study the action of falling water in view of the fact that banks on either side were suggesting different type of protecting works than those for the central deep channel. The river bed being rocky, peagravel was used below downstream works for scour study instead of sand to provide for adequate roughness in the model. The side subsidiary weirs were kept ten feet high so that the whole side discharge was diverted on to the deep central channel. This also ensured better standing wave formation at the low floods.

RAO BAHADUR JOGLEKAR introduced item (2) and said that as a result the experience gained in the working of the Sukkur Barrage and its models, the investigation for improving the design of the Kotri Barrage prepared by the Sind officers was subjected to a very intensive and detailed study in models.

MR. GOLE said that the experiments had been carried out in continuation of those which were dealt with in their Annual Report (Technical) for 1946. The main point covered were (*a*) elevation of sills of head regulators ; (*b*) the design of the left and the right pocket and the optimum number of spans, (*c*) the relative merits of a short pocket *versus* a long pocket; (*d*) the design of guide banks; (*e*) the location of the left abutments; (*f*) design of right and left pocket divide walls and (*g*) conditions to be expected after the operation of the barrage. Studies made of each of these items were described in details.

Introducing item (3) MR. GOLE said that all the experiments for the Lower Sind Barrage at Kotri were carried out in such in way that there was no imposition on the regulation of the left and right arms of the Barrage.

MR. KUTTIAMMU introduced items (4) and (5).

In the absence of Mr. Hardiker item (6) was taken as introduced.

DR. MALHOTRA introduced item (7) and Dr. UPPAL introduced item (8).

Referring to item (1) MR. KUTTIAMMU said that he would like to suggest a few improvements for the consideration of the designers and research officers. The crest width of the dam was proposed to be ten feet. He did not know it if

was intended to allow cars to go over the dam. If not it could very much be reduced with advantage. In the alternative a bridge or a foot bridge supported on peirs could be considered. By reducing the width of flat portion at crest and by adding an upstream lip, the coefficient of discharge could be increased by about 35 per cent. and the length reduced proportionately. With the present length the spillway extended too much into the margins of the river, and by reducing the length this could be avoided and cost of downstream productive works reduced.

As regards the downstream protective works, the stilling basin obtained by model experiments in 1946 would seem to be quite good. But the construction staff had found some difficulties in its adoption. The present design for the central portion consisted of a subsidiary weir three feet high at a distance of 90 feet from the toe of the dam and a baffle two feet high 40 feet from toe. From their experience on models of this type, they might be able to economise further on this design. The baffle might be replaced by a single row of blocks. It was possible, that these blocks might give all the benefits of the baffle and more, and might prove to be cheaper. Blocks four feet by two feet spaced eight feet clear apart at a distance 1/3 length of basin (30 feet in this case) might, perhaps, be all right.

MR. GOVINDA RAO enquired from Mr. Gupta whether the method adopted for the dissipation of energy was by turbulence or by creating a standing wave, and said that in the former case there was a danger of retrogression of bed levels.

MR. GHOTANKAR referred to item (I) and remarked that in the experiments a co-efficient of 3·07 was obtained in a 1/25 model. The co-efficient was expected to be higher for the prototype as the weir was narrow-crested and had L_2/D-0·9 against 2·0 required for broad crested weirs. He further enquired the maximum discharge that had been observed at this site and asked if the shape of the catchment was taken into account when working out a discharge of 68,000 cusecs according to Inglis formula. He further enquired as to why the flood absorption capacity was not considered as this in his opinion would have had a very important bearing in reducing the out-going discharge. With these corrections it would have been possible either to reduce the length of the weir or increase the full reservoir level. Further he remarked that it was not clear why such an intricate protective device was proposed when rock was exposed on bed downstream of the weir.

MR. GUPTA replying to the criticism said that the width crest of the Nagwa Dam was kept ten feet because of the insistence of the construction engineers on the site who felt that a width less than ten feet would not be suitable for inspection purposes. It was feared that blocks would get easily detached from the rocky bed in comparison to one continuous baffle, hence later device was recommended. As regards Mr. Govinda Rao's query the energy was dissipated by creating a standing wave. Replying to Mr. Ghotankar, he said

that the maximum flood discharge observed at site in 1946 was 63,000 cusecs. The shape of the catchment must be taken into account by the construction engineers who used the Inglis formula. The actual maximum flood discharge worked out, however, resembled very closely to that of the Inglis. They had taken into account the flood absorption capacity as already mentioned in their last year's annual report. The nature of the rock being poor sand stone, the cheapest and effective preventive measures were suggested.

Dr. Bose referred to item 3 and remarked that in view of the shoaling across the right guide bank the right undersluices were likely to be masked.

Mr. Gole in reply said that the conditions were tested under worst hypothesis and due to the attraction of the central island a channel would persist round the central pitched island as shown by experiments and the conditions were observed to be equally good thus insuring correct curvature of flow for the right bank feeder.

Dr. Uppal also made some remarks.

Rai Bahadur Kanwar Sain said that in his opinion for practical considerations of construction the sloping glacis should be straight for all conditions of flow.

Mr. Malhotra in reply said that in the Punjab the profile of the glacis had generally been kept straight but in the case of Nangal Barrage the maximum and minimum conditions of flow warranted that it should be parabolic. This according to him was a peculiar case.

Dr. Malhotra referred to Montagu's profile and said that he had proved it to be based on a fallacy. He further added that the trajectory method which had been employed in the Punjab Central Designs Office regarding the glacis was perfectly right for gated conditions.

Mr. Vadhera enquired about the efficiencies of gravity parabolic glacis profiles.

Dr. Uppal in reply stated that they had carried out a number of experiments on Montagu's as well as Dr. Bose's glacis profiles and that they had found that there was no difference between the two as far as their results showed and further in the construction itself there was very little difference.

Regarding the uplift pressures to be expected on the Nangal weir Rai Bahadur Handa said that when the design prepared by the Punjab Central Designs Office was examined by Dr. Malhotra he had suggested that the design assumption for reducing uplift pressures by 50 per cent. be verified otherwise for if it were not fulfilled the safety factor might be materially reduced. This had proved very prophetic and as the design developed it was considered that provision should be made for full uplift pressures.

DISCUSSION BY THE BOARD

The Secretary said that eight items were discussed at the Research Committee meeting (page 758). There was no resolution.

(iii) MATERIALS

PRELIMINARY NOTE

The following items were discussed at the 1947 Research Committee Meeting :

(1) Foundation rock of the Ramapadasagar Dam.

(2) Note on *surkhi* mortar.

(3) Sweating from Krishnarajasagar Dam.

Recent Literature.

(1) Concrete for dam mixed in transit—Construction Method, Vol. 29, No. 4, April 1947.

(2) Remote location complicates gunite repairs to dam—Construction Method, Vol. 29, No. 4, April 1947.

(3) Walter D. S.—Reclamation biggest earth fill dam—Engineering News Record, Vol. 139, No. 12, September 18, 1947.

(4) Whirler cranes build Bluestone Dam— Engineering News Record, Vol. 138, No. 4, July 24, 1947.

(5) Precast slabs put new face on old dam—Construction Methods, Vol. 29, No. 6, June 1947.

(6) McHenery, Douglas, Head, Structural Research Section, Bureau of Reclamation , Denver, Colorado, U.S.A—The effect of uplift pressure on the shearing strength of concrete. International Commission on Large Dams, Third Congress, Stockholm, 1948, R 48.

(7) Bourriot, Ingenieur E. N. P. C. France.—Essais d'adherence beton-rocher (Rock-concrete addesion tests) International Commission on Large Dams, Third Congress, Stockholm, 1948, R 39.

(8) Lea, F. M., O.B.E., D. Sc., F.R.I.C., Director of Building Research, Department of Scientific and Industrial Research.—Special cements for large dams—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 11.

(9) Hakanson, Per—General principles for a specification of special cements—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 14.

(10) Venables, F. de L., Research Engineer, Sydney, Australia—Experience in the use of low heat cement—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 16.

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(14) Langavant, Cleret de, Ingenieur E.C.P.—Emploi des ciments spectaux pour barrage en France. (Use of special cements as construction material for dams in France)—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 26.

(15) Hoon, R. C., M.Sc., Ph.D., Research Officer on Special Duty, Central Waterways, Irrigation and Navigation Commission, New Delhi (India)—The use of *Kankar* lime as building material for major engineering works—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 24.

(16) Langavant, Cleret de, La mesure de la chaleur d'hydratation des coinents par la methode thermos (Measurement of the heat of hydration of cements by means of the thermos method)—International Commission on Large dams, Third Congress, Stockholm, 1948, R 27.

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(19) Groner, Chr. F.—Experience on use of special cement for dams in Norway—International Commission on Large Dams, Third Congress, Stockholm, R 43.

(20) Steele, Byram W.—Concrete in large dams past, present and future—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 44.

(21) Meissner, Harmen S., Research Engineer, Bureau of Reclamation, Denver, Colorado, U.S.A.—Expansive cracking in concrete dams caused by reactive aggregate and high-alkali cement—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 47.

(22) Tyler, I. L., Manager of Field Research, Portland Cement Association, Chicago, R 11—A programme of cement research of significance to builders of large dams—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 50.

(23) Lalin, G. S.—Admixtures for the purpose of improving the workability of concrete—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 59.

(24) Kallauner, O., Prof. Dr. Ing.—Any remarks about tests of special cements.—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 60.

(25) Rawhouser, Clarence, Civil Engineer, Bureau of Reclamation, Denver, Colorado.—Temperature control of mass concrete to prevent cracking—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 61.

(26) Blanks, R. F., Chief Research and Geology Division, Bureau of Reclamation and Price, W. H., Head Materials Laboratories, Bureau of Reclamation—New developments in concrete and applications in the design and construction of concrete dams—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 62.

(27) Royen, N.—Report on actual observations of hydraulic structures built with low heat cement—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 63.

THE YEAR'S WORK

DISCUSSION BY THE RESEARCH COMMITTEE

There was no contribution under this sub-head.

DISCUSSION BY THE BOARD

The Secretary said that there was no contribution and no discussion at the Research Committee Meeting.

(iv) CONSTRUCTION

PRELIMINARY NOTE

There was no contribution or discussion under this head at the 1947 Research Committee Meeting.

Recent Literature.

(1) The Clark Dam—Construction methods and equipment—Commonwealth Engineer, Vol. 34, No. 6, March 1, 1947.

(2) Boswell, C. C. and Giesecke A. C.—Maintenance of heavy concrete structures Minnesota Power and Light Company Practice—Journal of American Concrete Institute, Vol. 17, No. 4, February 1946.

(3) Goodall, G. E.—Horizontal joint put in arch dam in effort to prevent cracking—Engineering News Record, Vol. 138, No. 26, December 26, 1946.

(4) Work resumed on Centre Hill Dam—Engineering News Record, Vol. 138, No. 16, April 17, 1947.

(5) The Dixence Dam, Switzerland—Science and Culture, Vol. 13, No. 4, October 1947.

(6) Fall River Dam—Construction Methods, Vol. 29, No. 7, July, 1947.

(7) The Clark Dam, Tasmania—Concrete and Construction Engineering, Vol. 42, No. 8, August, 1947.

(8) Walker, F. C.—Four earthfill dams create 1,873 acre Horsetooth Reservoir in Colorado—Civil Engineering, Vol. 17, No. 6, June 1947.

(9) Rapid construction of stirling earth dam—Commonwealth Engineer, Vol. 35, No. 2, September 1, 1947.

(10) The Cherokee dam—The Tennessee Valley Authority adds another handsome structure to the TVA chain—Arckitectual Forum Bostin.

(11) Central Utah project—Bureau of reclamation—Washington 1947.

(12) Blanchet, Ingenieur I.E.G. et E.I.H., Mesures de defermations sur le batardeau amont du barrage de Saint-Etienne-Cantales. (Deformation measurements on the upstream Coffer dam of St-Etienne-Cantales dam)—International Commission on Large Dams, Third Congress, Stockholm, 1948, C3.

(13) Burwell, Edward B., Jr., Chief, Geology and Geophysics Branch Office, Chief of Engineers, U.S. Department of the Army, Washington, D.C.— Foundation Engineering for Large Concrete Dams—International Commission on Large Dams, Third Congress, Stockholm, 1948, C 6.

THE YEAR'S WORK

The following items were discussed at the 1948 Research Committee Meeting :—

(1) Coffer dam for the Ramapadasagar Project.

(2) Ramapadasagar Dam—seepage into the excavation pits.

(1) COFFER DAM FOR THE RAMAPADASAGAR PROJECT(11)

ABSTRACT

The results of experiments on the 1/300 scale model of the Ramapadasagar Coffer dam were described in Annual Report for 1946. With the experience gained by the operation of the 1/300 scale model which formed the first series of studies and virtually served as a pilot model, a model to scale 1/150 was designed obviating the limitations of scale effects of the former model. It was considered that the larger model would simulate the prototype features more faithfully than the smaller. Gives results of experiments carried out in this long model.

DESIGN OF THE MODLE

Since this was a model of mobile bed in which flow pattern was the dominant feature a geometrically similar scale model in which Froude's law and Mannings frictional law were simultaneously satisfied was indicated. Consequently a model to scale 1/150 was designed since it was the largest that could conveniently be put in at Poondi. For moulding the bed sand of 24 to 30 mesh size (0·79 to 0·59 mm.) was used which was found to satisfy the frictional relationship to a fair extent. For the normal river condition without the Coffer dam the surface fall obtained in the model was about 3·17 feet per mile compared with the prototype value of about 2 feet per mile. There was neither sheet movement of sand for the normal river condition nor any injection of sand was made. But with the construction of Coffer dam scours were produced giving the indication sought for.

The scale ratios for the model were as follows :—

Linear scale	..	..	..	$1:150$
Velocity scale	..	..	..	$1:\sqrt{150}=12\cdot25$
Discharge scale	..	..	..	$1:(150)^{5/2}=275,600$
Friction scale	..	..	..	$1:(150)^{1/6}=2\cdot305$

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(11) Irrigation Research Station, Madras, Annual Report, 1947, pages 4-9.

CONSTRUCTION OF THE MODEL

The layout of the model shown in Figure 4 C.30 was generally similar to the 1/300 scale pilot-model. It was housed inside two large M.B. Sheds 110 feet × 35 feet each and sheltered against wind and rain. The water-supply was drawn by gravity from the reservoir through two pipes of 12 inches diameter and further supplemented by draw off from two static tanks with a combined capacity of one million gallons. The water was first admitted into a tank of 14 feet diameter in which the constancy of head was secured by providing a spillway of great length as shown in Figure 4 C.30. The supply from the constant head tank was drawn through a sluice into a masonry flume 3 feet wide × 4 feet deep and admitted to a standing wave measuring flume. The discharge from the standing wave flume could be either admitted to the model or diverted to waste as conditions required.

The model proper was enclosed inside masonry walls of 9 inches thick and constructed on the impervious floor of the sheds. It consisted of a fore-bay with stilling racks, a tail bay, tilting tail gate and other usual appurtenances. The model represented a length of 7,000 feet upstream of the dam line and, 5,500 feet downstream. The non-erodable margins, the Central Mahanadi Hill and a reach at the commencement of the model between 7,000 feet and 5,000 feet were moulded rigid. The water surface elevations over the model were tapped by porous pressure boxes embedded underneath the river bed at points 4,000 and 1,000 feet upstream and 2,000 feet downstream of dam line in the positions marked in Figure 4 C.30. The water level from the pressure boxes was communicated through 1/2 inch diameter brass tubes to a battery of guage pots mounted on a stand. From each of the connecting pipes a branch was taken off by a "Y" connecting piece and the pressure communicated to manometer tubes mounted on a stand as a battery. The manometer tubes enabled sucking out air from the connecting tubes though they could also be used to record the surface elevations by precision instruments as a cathetometer. Normally the water levels were all read in the gauge pots by a sliding pointer gauge reading to $\frac{1}{1,000}$ foot. From these readings the afflux and surface fall were computed.

Bench marks were located on the side walls at each 1,000 feet cross-section to serve as reference points for moulding the bed, *etc.* The bed was moulded by a system of female templates. At various points in each cross-section a number of base blocks were embeded with their tops at such a level that by placing a tubing of nine inches length vertically on the top of the block the top of the tubing could indicate a point on the corss-section of the bed. Each block had a pin projecting up to engage the tubing. After moulding the bed as per ordinates indicated by the tubings, the tubes could be removed so as not to function as obstacles to flow in the scoured bed.

There was a trolley built up of warren girders and moving on tram rails along the parallel sides of the model walls, to facilitate tracing of surface and bed flow lines, scour profiles and other observations.

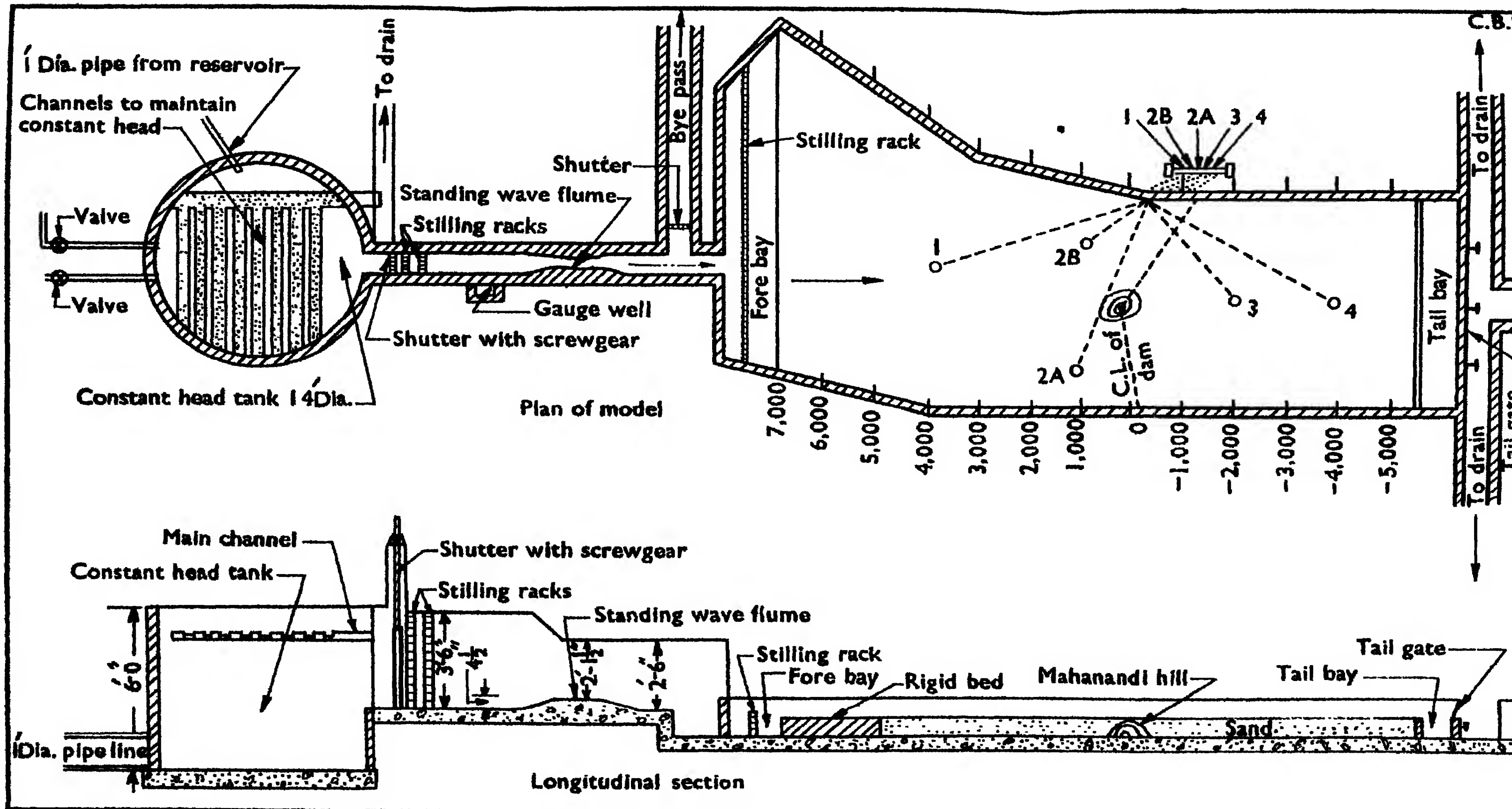

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Figure 4C.30:- *Showing layout of Ramapadasagar Cofferdam Model*

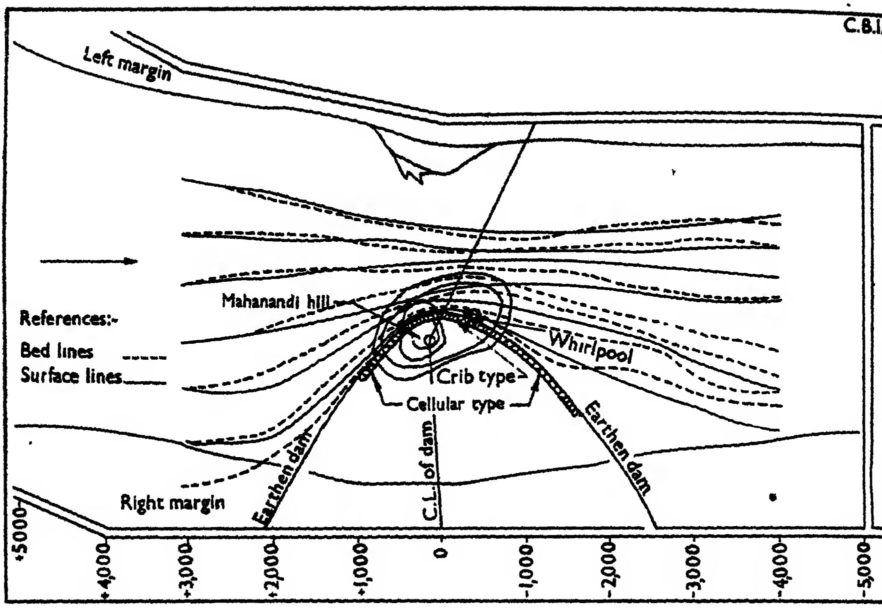

Figure 4C.31:- *Showing flowlines at bed and surface with west side cofferdam (Ramapadasagar cofferdam model)*

1,000 500 0 1,000 2,000 3,000
Scale Feet

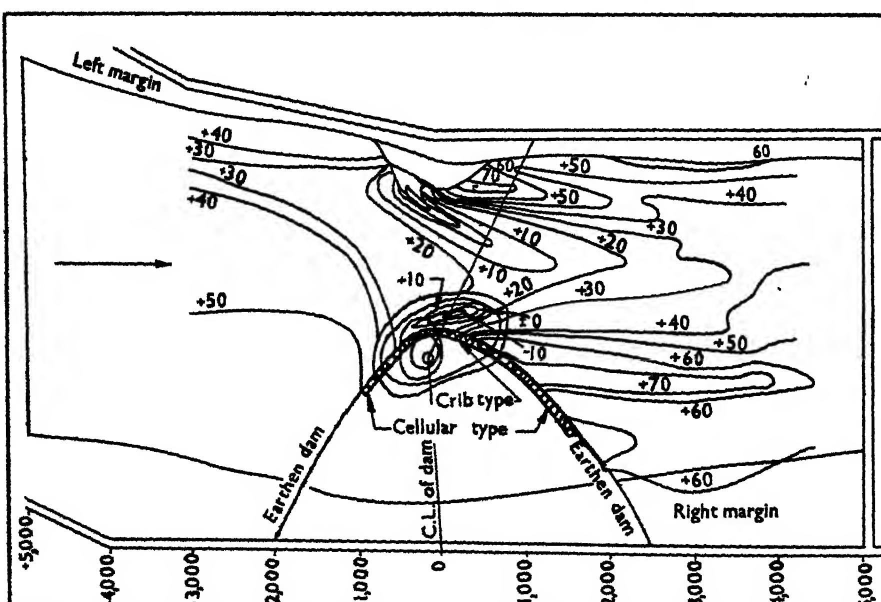

Figure 4C.32:- *Showing scour contours with west side cofferdam (Ramapadasagar cofferdam model)*

1,000 500 0 1,000 2,000 3,000
Scale Feet

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OPERATIONS OF THE MODEL

(*a*) The experiments conducted could be described under two groups: those with the normal river and those with the West Coffer dam in position. In the first group the model was first filled with water at a slow rate with the tail gate in the raised position. The model discharge corresponding to 2·25 million cusecs was admitted to the standing wave flume and the water diverted to waste till a steady discharge was attained. It was then passed on to the model. The tail gate was lowered till the gauge reading at 2,000 feet downstream of dam line corresponded to that of the prototype for that discharge. The gauge levels at other points were all recorded from which the surface fall and afflux could be worked out. In the subsequent experiments discharges corresponding to 1·85, 1·35 and 0·91 million cusecs were passed and the observations repeated. There was no appreciable bed movement of sand in these experiments as has been already stated.

(*b*) In the second group of experiments the river bed was moulded as per normal cross-sections and the West Coffer dam was built in position embracing the Mahanadi Hill at the nose, *vide* Figure 4 C-31. A discharge corresponding to 2·25 million cusecs was passed. The paths of bed lines and surface lines were traced by wool threads and plotted, *vide* Figure 4 C-31. At the conclusion of the experiment which lasted 5¾ hours the pattern of the scour was taken and plotted *vide* Figure 4 C-32. For this the water was emptied in stages when the boundary of the water edge furnished a contour corresponding to that level. By reading the co-ordinates of points on the water edge the contours were plotted in the sheet directly.

OBSERVATIONS

(*a*) The results of observations of water surface elevations are furnished in Table 4 C-11. It may be noted that the values were generally higher than those obtained on the $\frac{1}{300}$ model. The surface falls too were higher than in $\frac{1}{300}$ model. This was to be expected on account of the higher friction in this model than in the $\frac{1}{300}$ model. For a more correct simulation of friction the bed material should be replaced by a finer type.

(*b*) The results of experiments on the $\frac{1}{120}$ scale model with the West Coffer dam in place are shown in Table 4 C-12. It may be seen that in this case too the afflux noted was higher than in $\frac{1}{300}$ model.

(*c*) The scour pattern was similar to that obtained with $\frac{1}{300}$ model. In the $\frac{1}{120}$ model a series of powerful whirlpools were noticed which in the smaller model were inconspicuous on account of viscous effects. The larger model was generally better simulating the prototype features. Near the nose the maximum scour noticed extended to—20. But in this region rock is available at high elevation and there would be no danger to the foundation of the coffer dam. The alignment of the coffer dam appeared to be generally satisfactory.

(d) The flow pattern obtained is shown in Figure 4 C-31. It indicated that the upstream face of the coffer dam would act to accelerate the flow gradually and deflect the current away from the coffer dam at the nose. Eddying resulted and consequent exposure of rock stratum near the nose. A tendency to shoaling a little downstream of the nose owing to the repulsion of the bed line from the right bank in this reach was also indicated which is also confirmed by Figure 4 C-32.

CONCLUSION

(a) These rapid tests indicated that the $\frac{1}{150}$ model was able to simulate prototype conditions better than the $\frac{1}{300}$ pilot—model. Scale was sufficiently large to enable the various effects to be carefully studied.

(b) The values of afflux obtained were higher than those observed on $\frac{1}{300}$ model. Friction on this was obviously greater and it was evident that by altering the bed material and obtaining correct adjustment of friction or by adopting some other model device (such as tilting), reliable values could be obtained.

(c) The alignment of the West Coffer dam appeared satisfactory. There was eddying and scouring of bed near the nose, but as this was founded on rock (Mahanandi Hill) no danger could happen. There was no rapid current and no bad eddying near the cellular piles—no scours developed near them. Main scouring of bed occurred in the centre of the stream on the left as observed on the 1/300 model.

TABLE 4 C-11

Discharge in million cusecs	Condition	Model	Water levels				Surface fall in feet/mile	Prototype fall in feet/mile
			No. 1 at 4000 down-stream	2-A at 1000 up-stream	2-B at 1000 up-stream	No. 3 at 2000 down-stream		
2·25	Normal	1/150	93·6	92·5	92·1	90·0	3·17	2·0
,,	Do.	1/300	92·7	91·2	..	90·0	2·38	,,
1·85	Do.	1/150	91·7	90·6	90·6	88·7	2·60	1·93
,,	Do.	1/300	90·8	89·6	..	88·7	1·85	,,
1·36	Do.	1/150	88·7	88·1	88·1	87·2	1·30	1·82
,,	Do.	1/300	88·0	87·7	..	87·2	0·7	,,
0·91	Do.	1/150	84·5	84·2	84·2	83·8	0·62	1·74
,,	Do.	1/300	85·1	84·8	..	84·2	0·79	,,

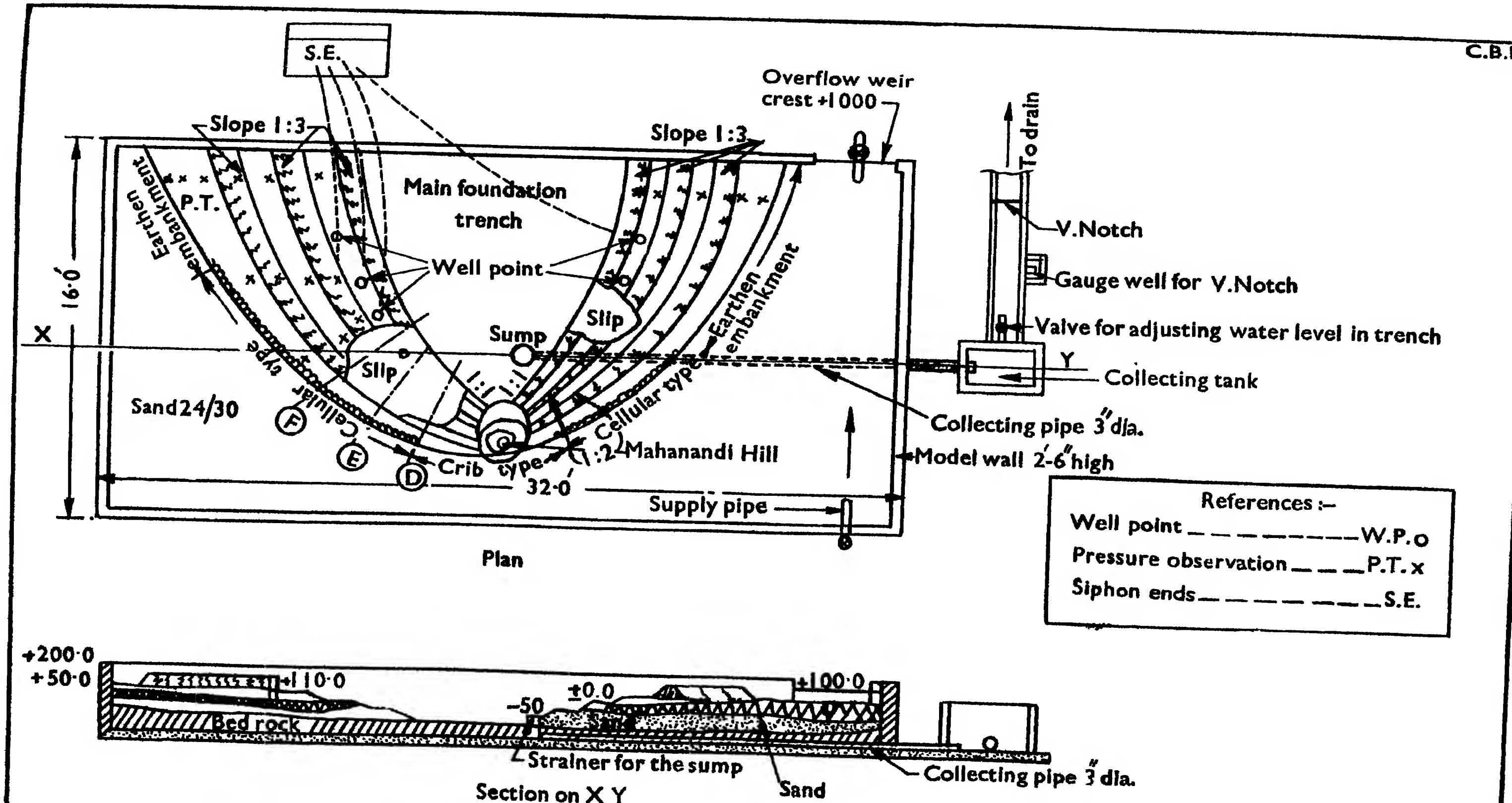

Figure 4C.33:-*Showing layout of three dimensional seepage model (Ramapadasagar Project-West Side Cofferdam)*

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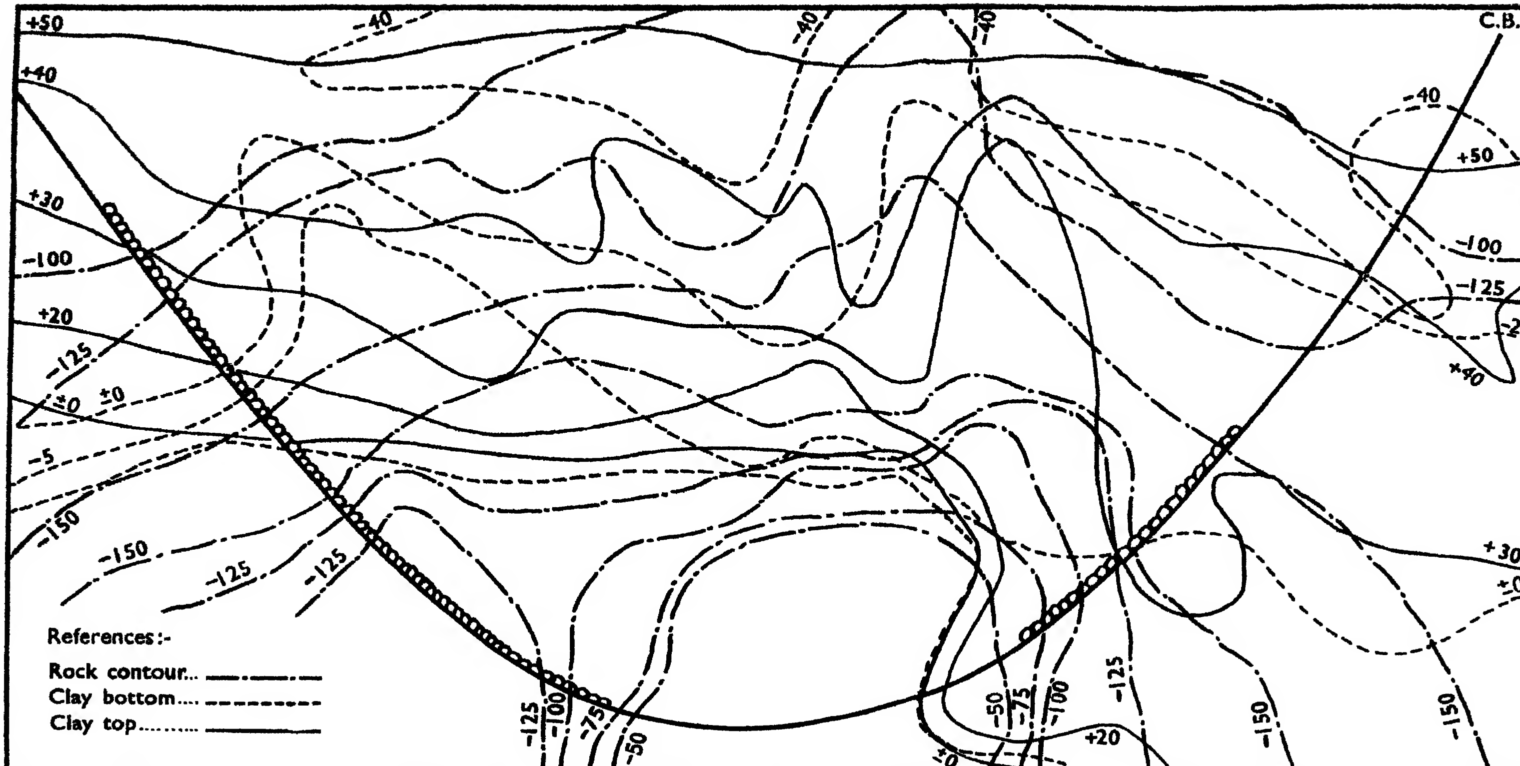

Figure 4 C.34:- *Showing rock and clay contours along west side cofferdam trench (R.P.S. Project)*

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TABLE 4 C.12

Discharge in million cusecs	Condition	Model	Water levels				Surface fall in feet/mile	Prototype fall in feet/mile	Afflux
			No. 1 at 4,000 up-stream	2-A at 1,000 up-stream	2-B at 1,000 up-stream	No. 3 at 2,000 down-stream			
2·25	West Side Coffer dam	1/150	97·6	Gauge not functioning	94·7	90·0	6·888	2·0	4·7'
		1/300	97·2	94·2	..	90·0	9·33	2·0	4·2'

(2) RAMAPADASAGAR DAM—SEEPAGE INTO THE EXCAVATION PITS [12]

ABSTRACT

The studies described in Article IX of the Annual Report of 1946 were made on sectional models and with the impervious rock assumed to be at—100. The limitations inherent in the studies were recognized and when sufficient data became available regarding the levels of rock and clay at the dam site as a result of extensive penetrometer and boring tests, it was considered desirable that further studies should be taken up with the aid of a full three dimensional seepage model. A model to scale 1/50 of the West Side Coffer dam and Trench was accordingly constructed and worked. During the course of operation of this model the necessity for repeating two dimensional model studies with typical sections where clay and rock are high became evident and the tests were, therefore, carried out. The data obtained from these sectional models were next incorporated into the full model and a satisfactory arrangement of slopes, well points, *etc.*, was obtained.

THE THREE DIMENSIO NAL MODEL

The model which wa s to scale 1/150 was enclosed within a masonry cistern 32 feet × 16 feet × 3 feet a nd reproduced the rock and clay contours, the masonry and cellular Coffer da m and a part of the river. The layout is shown in Figure 4 C.33 and the con tours of rock, clay bottom and clay top are separately shown in Figure 4 C.34. Both rock and clay were treated as impervious and made of masonry in the m del. The portion of the Coffer dam founded on the Mahanadi Hill was of masonry and the circular cells in continuation of this on either end were sheet piles. These wells in the model were made of iron sheets

(12) Irrigation Research Station, Madras, Annual Report, 1947 pages 103-113.

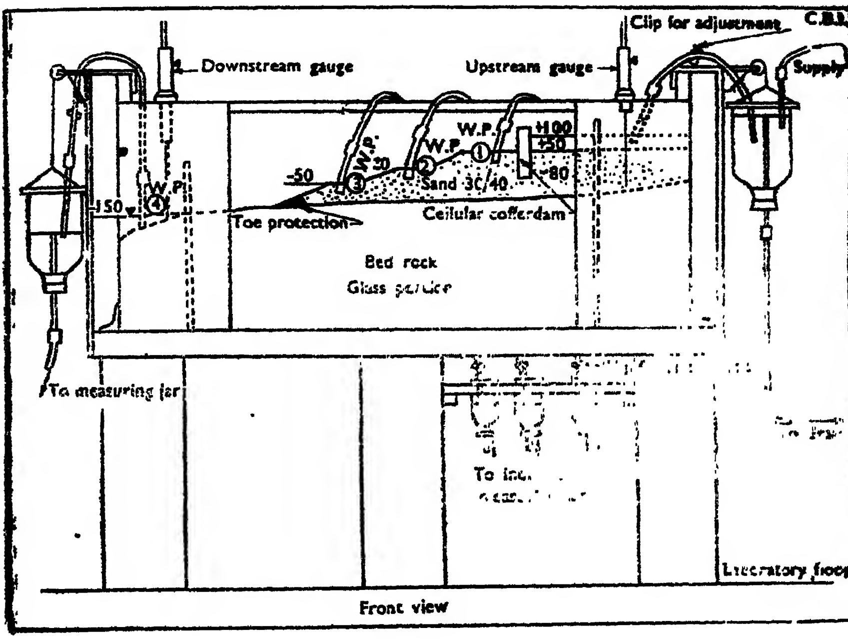
bent to form cylinders of 0·4≡60 feet diameter and were taken down to —60·0. The junction between any two wells was made water tight by putty. On the flanks the prototype will have earthen embankments which in the model were made of masonry. Sand passing through 24 mesh per inch and retained on 30 having a permeability coefficient of 16·8 feet per hour was used to represent the river bed. This, though more porous than the Godavari sand whose permeability is 11·88 feet per hour, was adopted on account of its availability in large quantities and as it was less likely to give capillary errors or to get choked during the course of experiments than a finer grained sample. The slopes in the foundation trench were cut accurately. Water on the upstream was let in through a 1½ inches hose pipe and was maintained at + 100 for all experiments by means of proper controls. From the bottommost part of the trench a 3 inches pipe was taken and connected to a cistern outside the main cistern. This had a control valve by operating which water could be withdrawn from the trench and maintained at any desired elevation. The water so withdrawn was measured over a V-notch lower down. The saturation gradients underneath the slopes of excavation were detected and measured by means of a copper pointer attached to an electrical circuit which included a galvanometer. One end of the circuit was connected to the water upstream in the model. By introducing the pointer into a glass tube buried on the slope it could be brought into contact with seepage water. A deflection in the galvanometer indicated the contact.

In the later experiments on the model a number of well points were established on the slopes. These consisted of cylindrical strainers 2 inches diameter sunk 6 inches deep into the sand. Water in these was withdrawn by siphoning by means of rubber tubes. The discharge in each tube was maintained steady by proper overflow arrangements at the tail-end and was measured quantitatively.

FIRST EXPERIMENT ON THE MODEL

The model was first worked without any well point on the slopes, the upstream water level was maintained at +100 and downstream water level was maintained successively at various levels ranging between + 70 and — 150. For all these conditions the following two defects were observed : (*a*) The toe of the sand slope started slipping whenever the downstream water level was low, (*b*) boiling occurred in the regions shown in Figure 4 C.33. Both these had a cumulative effect and developed steadily and finally covered the entire sand slopes resulting in their complete failure. The toe slips could be obviated by rip rap protection. The boils on the slopes had to be prevented by changing the

slopes, or by lowering the seepage gradient by pumping. The presence of impermeable clay at high level (which was not reproduced in the original sectional model) was evidently responsible for the boils. To study this more thorougly three typical sections D, E and F in Figure 4 C.33 were taken and tested on sectional models.



: ..Figure 4 C.35 : *Showing arrangements of experiments on sectional seepage model (Ramapadasagar Coffer dam).*

SECTIONAL MODELS

These models were constructed one after another to a scale of 1/400 inside a seepage tank 5 feet long × 6 inches wide × 2 feet deep. The set up is shown in Figure 4 C.35. The front side of the tank was glass which facilitated visual observation of seepage lines. The cross-section selected for study was faithfully reproduced in the model. The sandy bed was reproduced in sieved sand of 30—40 mesh having a co-efficient permeability of 30·3 feet per hour. Clay, rock, coffer dam section, *etc.*, which were assumed impermeable were all put in cement. There were arrangements for controlling upstream and downstream water levels for siphoning out water from each of the well points introduced on the slope and for measuring the water collected at each outlet.

EXPERIMENTS ON SECTIONAL MODELS

Several runs were made with these models. In all these upstream water level was maintained at +100 and downstream water level at—150. When the well points were not functioning there were slips on the slope. These could be prevented by depressing the saturation gradients by siphoning out water at the well points. Several combinations of sections were examined in order to determine the best arrangement for each condition, namely, the arrangement which involved the least consumption of power for pumping. The results are all furnished in Tables 4 C.13, 4 C.14 and 4 C.15.

SECOND EXPERIMENT ON FULL MODEL

With the information thus gathered, observations on the three dimensional model were resumed. Toe protection was provided for the sand slope cutting and well points at various positons were operated. The slopes remained stable and by the judicious operation of the well points the saturation gradient could be depressed to the extent necessary and a most economic consumption of power for dewatering ensured.

DISCUSSION OF RESULTS

We have already observed that toe protection and pumping underneath the slope are both necessary for maintaining the slopes of excavation stable. A reference to Tables 4 C.13, 4 C.14 and 4 C.15 gives some indications of the economics of pumping. From Table 4 C.13 it may be seen that the minimum power was consumed when pumping was resorted to at lower levels, *viz.*, pumps 3 and 4. When pumping was restricted to 1 and 4 the power consumed was the maximum. This was in spite of the fact that pumping at lower levels involved considerable lifts on the delivery side. In the higher elevations the amount of pumpage required for obtaining a small depression was overwhelmingly large. The best arrangement will be to have the well points in the regions where floatations may develop : by this arrangement the saturation gradients can be kept well below the safe limits. Obtaining the same result by pumping higher up is not economical. In the sectional model in which bed rock was kept at—200 and downstream water level at —70 and upstream water level at +100·0, we had seen that seepage (prototype) per foot run of the Coffer dam was 0·14 cusecs. Observations made on the three dimensional model in the first experiment, *i.e.*, before the well points were introduced indicated a total seepage of 307·6 cusecs (prototype). In the second experiment, *i.e.*, after the introduction of well points along the slopes considerable change in the total seepage was observed. A closer and more detailed study on this is necessary and will be made in due course.

CONCLUSIONS

(*i*) In determining the stability of the slopes of the excavation against slips the rock and clay contours underneath the sandy surface have a marked influence when they are near the surface.

(*ii*) In the West Coffer dam Trench the slopes cannot be maintained stable by merely giving a flat slope or by providing berms. Pumping at various sections along the slopes should be resorted to.

(*iii*) Pumping from wells at elevations where boils may appear is essential for depressing the saturation gradients. Pumping at higher elevations will be unnecessary and uneconomical.

(*iv*) Toe protection is essential for the entire length of the excavated slope.

(*v*) Seepage as calculated from the sectional models works out to 0·14 to 0·07 cusecs per foot length. The total seepage for the full trench as obtained on the three dimensional model is about 3[illegible]0 to 400 cusecs. Assuming this entire seepage to have occurred within the length (2,550 feet) covered by the cellular Coffer dam the average seepage per foot run works out to 0·12 to 0·16 cusecs.

TABLE 4 C.13

Condition				Quantity pumped					Level of water at each well points			
Experiment number	Upstream water level	Downstream water level	Pumping from	At No. 1	At No. 2	At No. 3	At No. 4	Total for 200 ft. length 1+2+3+4 c/s	At No. 1	At No. 2	At No. 3	Remarks
1	2	3	4	5	6	7	8	9	10	11	12	13
1 ..	100·0	—150·0	1 and 4	18·0004	..	..	6·596	24·600	—20·0	—50·0	—87·0	No slips.
2 ..	100·0	—150·0	1, 2 and 4	11·772	1·864	..	4·576	18·212	—33·0	—60·0	—95·0	Do.
3 ..	100·0	—150·0	2, 3 and 4	..	5·088	2·352	4·708	12·148	—7·0	—53·0	—88·0	Do.
4 ..	100·0	—150·0	3 and 4	..	..	3·952	5·556	9·508	—2·0	—37·0	—85·0	Do.

N.B.—Values corrected to Godavari sand using the value of (*b*) given by Physics and Soil Mechanics Officer for 30/40 sand.

TABLE 4 C.14

Experiment number	Condition: Upstream water level	Condition: Downstream water level	Condition: Pumping from	Quantities pumped: At No. 1	Quantities pumped: At No. 2	Quantities pumped: At No. 3	At No. 4	Total 1+2+3+4	Level of water at each well point: At No. 1	Level of water at each well point: At No. 2	Level of water at each well point: At No. 3	Remarks
1	2	3	4	5	6	7	8	9	10	11	12	13
1 ..	+100·0	—150·0	Trench (4)	Nil	Nil	Nil	17·430	17·430	+15·0	—12·0	[illegible]	Slips started at terrace No. 3, vide Fig. 76.
2 ..	+100·0	—150·0	(1) and ([illegible])	14·126	Nil	Nil	[illegible]	[illegible]	—12·0	—70·0	[illegible]	There were no slips.

TABLE 4 C.15

Experiment number	Condition: Upstream water level	Condition: Downstream water level	Condition: Pumping from	Quantities pumped: At No. 1	Quantities pumped: At No. 2	Quantities pumped: At No. 3	Quantities pumped: At No. 4	Quantities pumped: Total 1+2+3+4	Level of water at each well point: At No. 1	Level of water at each well point: At No. 2	Level of water at each well point: At No. 3	Remarks
1	2	3	4	5	6	7	8	9	10	11	12	13
1 ..	+100·0	—150·0	1 and 4	17·04	..	..	6·004	23·04	6·50	—33·0	—66·5	No. slips
2 ..	+100·0	—150·0	1, 3 and 4	7·512	..	3·692	28·8	14·084	10·00	—40·0	—75·0	Do.
3 ..	+100·0	—150·0	1, 2 and 4	7·208	1·476	..	5·476	14·240	10·00	—33·0	—68·0	Do.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. KUTTIAMMU introduced items (1) and (2).

MR. GOLE remarked that in item (1) the scour pattern was stated to govern the selection of a geometrically similar model in preference to a vertically exaggerated model. The scour at the Coffer dam would actually depend on the flow conditions of the river which necessitated a vertically exaggerated model. Coffer dam experiments were carried out for the Lower Sind Barrage at Kotri and it was found necessary to have a vertically exaggerated model to reproduce the correct flow conditions.

MR. GHOTANKAR said that on page 9 of the Madras Report it had been stated that the whirl-pools were inconspicuous in the smaller model because of viscous effects. Actually the Reynolds' Number with the lowest and the highest discharges roughly worked out to 8.000 and 17.000 respectively which was well above the critical value and so the viscous effects did not prevail. The correct reason for non-reproduction appeared to be the adopting a geometrically similar model in place of a vertically exaggerated model. In such cases where there were rigid structures requiring geometrical similarity in one and the model requiring vertical exaggeration the best way was to effect a compromise as had already been proposed in the Annual Report for 194[illegible] of C.W.I.N.R.

RAO BAHADUR JOGLEKAR remarked that where curvatures had to be reproduced, vertical exaggeration was absolutely necessary.

DR. UPPAL referring to the vertical distortion said that Dr. Bose would bear him out that this was a very slippery matter because no matter how small the distortion was, it was difficult to control and the conclusion could not be stated as definite.

MR. KUTTIAMMU in reply said that they had carried out the geometrically similar model test but they found that they could not reproduce the bed movements. The work was still in progress. They had considered suggestions for vertically similar model but due to the limitations of space and discharge they could not adopt it.

DR. BOSE said that there was no rigid hard and fast theory about the vertical distortion of models and that some sort of distortion in the model was sometimes found to be necessary. Referring to the slopes for the normal river condition without Coffer dam he said that the surface fall obtained in the model was about 3·17 feet per mile compared with prototype value of two feet per mile. This led to the conclusion that some part of distortion was necessary which should give similarity to natural conditions.

It was decided that the subject should continue to remain on the Agenda.

DISCUSSION BY THE BOARD

THE SECRETARY said that two items were discussed at the Research Committee Meeting (page 797). There was no resolution.

5C. Dam Appurtenances

PRELIMINARY NOTE

This subject was introduced for the first time in 1947 with the following sub-heads :—

(i) Inlet and outlet works (ii) Tunnels (iii) Conduits and penstocks, (iv) Fish Passes (v) Other works.

(i) Inlet and outlet Works

There was no contribution or discussion under this sub-head at the 1947 Research Committee Meeting.

Recent Literature.

(1) Warnock, J. E. and Pound, H. J.—Coaster gate and handling equipment for river outlet conduits in Shasta Dam—Transactions of the American Society of Mechanical Engineers. Vol. 6?. No. 3, April 1946.

(2) Ball, J. W. and Herbert, D. J. Hydraulic Engineers, Bureau of Reclamation. Branch of Design and Construction, Denver, Colo., U.S.A.—The development of High—Head Outlet Valves—The Second Meeting of the International Association for Hydraulic Structures Research. Stockholm, 1948 Paper No. 2?.

THE YEAR'S WORK

The following items were discussed at the 1948 Research Committee Meeting :—

(1) Ramapadasagar Dam—river outlets and density currents.

(2) Bumping of plus 12 gates in Krishnarajasagar Dam.

(1) RAMAPADASAGAR DAM—RIVER OUTLETS AND DENSITY CURRENTS (1)

The design of the Ramapadasagar Dam was altered introducing a larger number of sand sluices and reducing the spillway length. The new and final design consisted of the following :—

Spillway—16 vents of 135 feet × 28 feet each.

Sand sluices—10 feet × 20 feet—160 Numbers.

Sill of sand sluices— +93·0.

Experiments were taken up on a model of this design with a view to test its performance in the several aspects of design and specially in the disposal of density currents. A method was evolved after a series of trials of producing "density currents" whose characteristics could be measured and recorded.

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(1) Irrigation Research Station, Madras, Annual Report, 1947, pages 96-97.

The model was constructed to scale $\frac{1}{100}$ and was fitted up in the three feet wide glass flume. The model was fitted with its upstream face 15 feet away from the flume head. This enabled good examination to be made of the currents approaching the structure, by visual observation through the glass sides of the flume. Sand was packed downstream of the model to represent river bed on which nature and depth of scours could be observed.

Near the head of the flume was kept a large drum on an elevated platform. A mixture of locally available red earth with water in known proportions slightly coloured by the addition of potassium permanganate was kept in the drum. Glass and rubber tubes taking off from this drum were dipped in the flume and by siphonic action the dense solution could be injected into the water at the place and time desired. The solution issued out in the form of a cloud and slowly moved towards the dam model. Its speed, lines of flow etc. could all be observed and recorded.

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(2) BUMPING OF PLUS 12 GATES IN KRISHNARAJASAGAR DAM (2)

Two sets of scouring sluices have been built in the Krishnarajasagar Dam. One set is located at 12 feet and the other at 50 feet above the average bed of the river. The first set consists of eight sluices, each vent measuring six feet in width by twelve feet in height. They are generally not operated when the depth of water over their sill exceeds 80 feet.

The plus 12 sluices though in operation from 1914, have not been observed to throw out any silt. Soundings taken in summer 1945, do not also indicate accumulation of silt in front of them. There is also practically no deposition of silt at the junction of the river with the reservoir 25 miles upstream of the dam. These may be due to the fact that silt is all held up by five anicuts above it.

The sluice vents are all rectangular in shape with no bell-mouth openings. Immediately after the gates which have been installed 10 feet from the front face of the dam, a drop 0·56 foot has been given in the floor of the tunnel. The vent opening is spanned over in rear of the gate with semi-circular masonry arches. Each vent is fitted with Stoney's free roller patented type gate.

These gates have been in service for nearly 30 years. Almost the several defects noticed in their working the gates when raised eight feet above sill upto 10 feet are found subjected to very heavy vibrations and are alternately thrown forward and backward. This phenomenon is called the bumping or banging of the gates. This causes heavy vibrations in the body of the

(2) Hydraulic Research Station, Krishnarajasagar, Annual Report, 1947, pages 19-25.

dam, producing an awesome chattering sound. This was brought to the notice of the manufacturers Messrs. Ransomes and Rapiers. After studying their working, Sir Wilfred Stokes, their Managing Director suggested the following remedies :—

(*a*) To bell-mouth the entrance to the culverts either by cutting away the masonry, or the addition of an upstream bell-mouth projecting into the reservoir. Both these proposals seem to be costly and difficult.

(*b*) The formation of an air vent in the skin of the sluice say, of a gauge 12 inches by 3 inches closed on the upstream face by a hinged flap arranged so as to be water-tight.

(*c*) Another treatment of the difficulty would be the provision of two or more bars projecting beyond the bottom of the gate which by reason of the stream [illegible] would apply [illegible] force to the gate (in the [illegible] back upstream until the gate [illegible].

These bars might [illegible] brought into the effective position [illegible] and when the discharge co-efficients had ceased to be of importance.

(*d*) An alternative method of dealing with this difficulty would be to provide bumping blocks or distance pieces which would restrict the movement of the gate up and downstream.

Unless the rollers and paths are already worn out unevenly there would not be any difficulty in applying this treatment. If, however, the wear is already considerable, it will not be possible to reduce the clearance enough to prevent bumping and without causing the new parts to stick when the gate is lowered to a position where the wear is less.

The suggestion of the firm to insert a gas pipe to study pressures was found to be impracticable as the space between the gate and gate castings was not more than 1/8 inch. There was also the probability of the pipe getting crushed due to water pressure and closing.

The Mechanical Engineer, in charge of the gates, in 1943, thought that the jet of discharge issuing from the sluice vent when the gate is opened to the critical height. sucks in air into the well, which while attempting to escape out shakes the gate standing in its way. He, therefore, proposed cutting in all entry of air by closing all the opening to the sluice well. This was tried without success in June 1947.

The present Executive Engineer (1947) in charge of the gate thinks the formation of the vacuum itself as suggested by Sir Wilfred Stokes is doubtful, as, when the gate is being raised continuously from its closed position, water from the reservoir rushes through the small clearance between the lintel piece and the skin plate of the gate and spills down on the top of the gate. This continues to happen till the top of the gate is below the landing in the masonry well on the upstream side. But when the gate top rises above the landing level water gets stored up on the landing for over two feet depth. This is a clear indication that the sluice vent in front of the gate is all filled up with water upto the lintel and even 8 feet above; and that no vacuum exists as apprehended. The bumping action of the gate is not, therefore, due to the vacuum in front of it. He had a doubt that the bumping of the gate may be caused by the roller frame lagging behind when the gate is being raised. The water hitting exposed portion of the roller frame below the gate may cause tilting. The remedy suggested was (1) fixing the roller frame end at the bottom to the rocking roller path so as to move with as little friction as possible, (2) deadening the velocity of the water at the roller frame by creating a chamber with shield plates on the exposed surfaces and (3) using deflectors in front of the side grooves to deflect away the current and leave the roller frame free from water action. The first one was found to be impracticable under the existing circumstances. The second suggestion has already been adopted for the other sluices supplied by the firm. The third suggestion of deflecting the current is not possible and it also lessens the co-efficient of discharge.

In July 1947, to find out if any relief from this bumping could be had by making the roller train move at the same rate as the gate, the gate was pulled out clear of the vent and also the roller frame. The gate and the roller frames were then tied together and lowered. No relief from bumping was observed.

The Director of the Hydraulic Research Station is of the opinion that the bumping of the gate is due to the turbulent action of the water in front of the gate which makes its effect felt when the gate is dipped in water for only about one foot or so.

An experiment was conducted with a steel damper screen being lowered to the front of the vent opening in one of the plus 12 gates. It was observed while raising the gate, that there was absolutely no vibration of any kind even after raising the gate beyond the lintel *i.e.*, 12 feet high. But while lowering it was found that the gate would not go down below the tenth foot at which level it began to jump up and down and the steel rope also came out of the sheave; with some effort, however, and making the gate move down suddenly, it was finally possible to lower the gate to the bottom. After final examination it was found that the plate was too long and that it projected into the vent a little beyond the sill and with the full flow of water, the plate practically acted as a fluid cill and prevented the gate from going down until the plate actually got bent and distorted. The experiment could not be repeated as the reservoir level rose above plus 92.

When [illegible] above the top of the gate through [illegible] between the gate and the lintel frame castings. An experiment was made closing all the openings of the sluice well preventing all access to air and raising the gate. The leakage water went on increasing till the gate was raised to 9·2 feet when it began to chatter. Suction of air and dust from the well also went on increasing till the gate was raised above 9·2 feet when the gate was clear of the jet of water shooting through the sluice tunnel. So also the leakage water stopped sprouting when once the gate was raised clear of the jet. This showed that as the *Vena Contracta* of the jet began to be defined more and more clearly as the gate was raised, the suction in the well began to increase indicating vacuum conditions in the space between the ceiling of the sluice tunnel and the surface of the jet of discharge. Bubbles of water could be seen breaking up as the leakage sprouted above, showing cavitation existed in the flow. The presence of pitted surfaces also confirms this view. In all gates operating under heads above about 25 feet (atmospheric pressure minus frictional losses), the water filaments will have to take very sharp turns when they are passing beneath the bottom of the gates, at certain critical heights of openings which are just insufficient to allow uniform stream lines to be formed. When the water filaments take very sharp bends cavitation has to be expected at those regions as the velocity will be very great and radius of turning very small. (Vr is constant) Pitting is therefore, invariably noticed in all gates operating under high heads. Where no bell-mouth openings have been given as in the present case, cavitation areas become large with consequent setting up of vibrations. The remedy seems to lie in giving correct bell mouth openings to gates. Steel damper plates are also helpful in minimising turbulence.

DISCUSSION BY THE RESEARCH COMMITTEE

Mr. T. P. Kuttiammu introduced item (1)

Mr. N. S. Govinda Rao introduced item (2) and said that the gates when raised eight feet above sill and up to ten feet vibrated violently. They were alternately thrown forward and backward. That phenomenon was called the bumping or banging or chattering of the gates.

Various suggestions were put forward, among others, some by Messers Ransomes and Rapier—the manufacturers, to remedy the defect, but so far none had succeeded in eradicating the trouble. The suggestions included (1) provision of holes in the gates, (2) provisions of a projected plate below the bottom of the gate.

In all gates operating under heads above 25 feet (atmospheric pressure minus frictional losses) the water filaments would have to take very sharp turns when passing beneath the bottom of the gates, at certain critical heights of openings which were just insufficient to allow uniform streamlines to be formed.

When the water filaments [illegible] high heads, cavitation had to be expected in those regions. Pitting was therefore, invariably noticed in all gates operating under high heads. Where no bell-mouth openings had been given as in the case under consideration, cavitation areas became large with consequent setting up of vibrations. The remedy seemed to lie in giving correct bell-mouth openings to gates. Steel damper plates were also helpful in minimising turbulence.

Further experiments, he said, were under progress.

MR. T. P. KUTTIAMMU said that Mr. Govinda Rao had supplied a very interesting and instructive note on "bumping of gates *etc*". They would eagerly watch the progress of his studies on the subject.

Study of the note showed that, no doubt, the reason for the bumping or vibration in the case was hydraulic and not mechanical. When the gate was slowly lifted, there would be shooting flow in the first instance which would develop into a submerged standing wave later on. There was no streamlining of the entry, the barrel inside was vaulted and there was no provisions for aeration in the upstream and downstream sides of the gate. All these were conducive to the development of abrupt changes in pressure and result in vibration. In modern designs the outlet was given a bell-mouthed entry and the barrel was made rectangular and uniform in section or slightly converging towards the downstream end and ample provision was made for aeration.

He would suggest that a large size model of the sluice be made with glass windows for observing flow conditions inside the barrel and plenty of piezometer tubes for obtaining pressure at various parts of the structure. Possible improvements could then be studied on the model. This should include remodelling of entry and barrel, and aeration on both sides of the gate.

DR. H. L. UPPAL said that $1\frac{1}{2}$ inch holes stopped chattering in Bonney Ville Dam gates.

DR. N. K. BOSE enquired as to how the vacuum was formed behind the gates.

MR. N. S. GOVINDA RAO explained the creation of vacuum by means of illustrations.

MR. M. L. SOOD suggested that a pipe could be fitted from the top which opened into the vacuum portion.

MR. N. S. GOVINDA RAO explained the difficulties in the working of such a pipe.

It was decided to retain the subject on the agenda.

DISCUSSION BY THE BOARD

THE SECRETARY said that two items were discussed at the Research Committee Meeting. There was no resolution.

(ii) Tunnels

PRELIMINARY NOTE

There was no contribution or discussion at the 1947 Research Committee meeting under this sub-head.

Recent Literature.

(1) Mutch H. W. Water for New Mexico desert—Western Construction News, Vol. 20, No. 9, September 1945.

THE YEAR'S WORKS

DISCUSSION BY THE RESEARCH COMMITTEE

There was no contribution under this subject and no discussion. (It was decided to retain the subject on the agenda).

DISCUSSION BY THE BOARD

THE SECRETARY said that there was no contribution and no discussion at the Research Committee Meeting.

THE CHAIRMAN RAO BAHADUR A. R. VENKATACHARI enquired whether there were any tunnels in Bhakra construction and what was their estimated cost.

RAI BAHADUR C. L. HANDA replied that 2 tunnels had been proposed. The cost of each tunnel including equipment would be about Rs. 3 *crores*, length 5,000 feet, diameter 50 feet—lined with 3 feet thick concrete. There was to be one tunnel on either side of the dam, the tunnelling was in sand stone.

(iii) Conduits and Penstocks

PRELIMINARY NOTE

There was no contribution or discussion under this sub-head at the 1947 Research Committee Meeting.

Recent Literature.

(1) Warnock J. E. and Pound H. J.—Coaster gate and handling equipment for river outlet conduits in Shasta Dam—Transactions of American Society of Mechanical Engineers, Vol. 68, No. 3, April 1948.

(2) HAMON, Ingenieur, A. M. and E. I. H., France—Essais de la conduite forcee No. 3 du barrage de l'Aigle (Penstock No. 3 tests at Aigle Dam). International Commission on Large Dams, Third Congress, Stockholm, 1948, R 41.

THE YEAR'S WORK

DISCUSSION BY THE RESEARCH COMMITTEE

There was no contribution under this subject.

It was decided to retain the subject on the agenda.

(iv) FISH PASSES

PRELIMINARY NOTE

This subject was brought on the agenda of the Research Committee in 1947. A detailed preliminary note was put up at that time. The Board approved the following resolution passed by the Research Committee last year.

> "Resolved that the Board is of the considered opinion that in the construction of protective river works, due consideration should be paid to the fishery resources of the waterways, by conducting faunistic and economic surveys of the fisheries. In view of the proposals to construct a large number of dams in the country the committee desires that irrigation research stations should undertake work on fish passses by associating fishery scientists with them in this work".

Further* at the 1947 Board Meeting, the Secretary explained that in February 1947 a letter was received from the Indian Council of Agricultural Research, stating *inter alia* that the Fish Committee (of the Indian Council of Agricultural Research) at its meeting held in October 1946 considered a note submitted by Dr. S. L. Hora, former Director of Fisheries, Bengal and recommended that the attention of the Central Board of Irrigation should be drawn to the necessity of foreign study of fish passes and the scientific subjects related thereto. The Council also supported the view. He requested the Board to consider the recommendation.

The President said that there was no doubt, that considerable work on fish passes had been done in England, America and Australia. Before, however, a man could be sent abroad, study be made of the work done already and of the specific problems awaiting solution. He should then be given a clear direction of what he should study abroad.

After some discussion it was decided that a note of the subject should be prepared in the Board's Office, and the Indian council of Agricultural Research informed of the action being taken.

A note has been prepared and the same will be placed before the members of the Committee.

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* Item XXVII under Administrative Agenda, 1947.

The following item was discussed at the 1947 Research Committee Meeting.

(1) Problems of fish cultivation in irrigation and hydro-electric reservoir-by Sunder Lal Hora, Director, Zoological Survey of India, Benares.

Recent Literature.

(1) Dams and Fisheries—Central Board of Irrigation Journal, Vol. 4, No 2, April 1947.

(2) Hora, Sunder Lal—Construction of dams and rivers fisheries—Central Board of Irrigation Journal, Vol. 4, No. 1, April 1947.

(3) Hora, Sunder Lal—Collection of papers on fish passes and fish control.

THE YEAR'S WORK

DISCUSSION BY THE RESEARCH COMMITTEE

There were no contributions under these subjects and no discussion.

It was decided to retain the subject on the agenda.

A BRIEF NOTE ON THE WORK OF FISH PASSES SO FAR DONE AND PROBLEMS AWAITING SOLUTION

BY

Captain P. R. Ahuja, P. S. E. I.

Deputy Secretary, Central Board of Irrigation.

DEFINITION

Fish-ways or fish passes are channels, series of pools or similar hydraulic structures constructed to aid fish in overcoming obstacles in migration. The obstacles can be natural, or artificial *e.g.*, weirs and dams.

A large volume of work in connection with the design and construction of fish-passes has been done in England, Scotland, Sweden, Belgium and America. The basic common requirements for any fish-ways are :

(*i*) The size of free cross-section to be sufficient to allow swimming movements considering the size and the number of fish using the fish-way.

(*ii*) The hydraulic conditions must be of such a nature that the passage of fish up the fish-way does not overtax the energy of the weakest individual fish.

(*iii*) The quantity and character of the flow and the placement of the fish way should be such that the downstream end will be easily accessible and also easily discovered by the fish.

(*iv*) The fish-passes should be able to withstand weathering, impact of floating logs, remain free from debris and excessive sedimentations, and further they should be easy to inspect and for carrying out repair works.

Modern fish-passes may be divided into the following types :

(1) The pool type, consisting of flight of stepped pools and often referred to as a fish ladder.

(2) The steep channel type.

(3) The fish lock.

(4) The fish lift or elevator.

Each of these has its advantage and disadvantages, and its own particular hydraulic problems.

In both pool and channel type of fish passes the main problem is the dissipation of the energy developed in such a manner as to provide smooth water with sufficiently low velocity of flow as will permit of the fish swimming against it in their passage upstream.

POOL TYPE

In the pool type of pass the pools form a series of steps ; the energy at each step must be so dissipated that there may be no appreciable velocity of approach at the opening to the next pool. The pools may be connected by notched overflow weirs or by submerged orifices.

The view is largely held that the larger the pools and the smaller the size between them ; the more efficient the pass will be whilst this is true to some extent, it forms an uneconomical basis for design. The pools of a fish-pass serve a double purpose. On the one hand they are intended to dissipate the energy generated by the fall, which spends itself in turbulence on the other to provide areas of slack water in which the fish can rest. Economy can be effected by designing the pools for fist purpose only and meeting the second by the provision of large pools at intervals. The investigations have showns that for energy dissipation a larger width of pool is unnecessary, and that by jet deflexion the length can be substantially reduced.

The pass with notched weirs is less suitable where considerable quantities of silt or fine sand are present in suspension as there is a danger of the pools becoming silted up. Again this is also less suitable where there is a considerable variation of water levels, between head and tail, or even at head alone. Under such conditions the pass with submerged orifices has got advantages over the weir type.

In rivers with gravel beds, there may be a tendency for the orifices to become blocked with gravel, and this has been regarded as a draw back to submerged orifices which otherwise present advantages over notches.

Extensive use is made of model experiments before evolving a fish pass for a particular site.

As a result of large scale extensive experiments on model fish passes of the pool type carried out in the Imperial College of Sciences and Technology South Kenzington, London from June to December 1945 by Dr. C. M. White, a design was developed incorporating a spoon shaped hollow in the floor of the compartments downstream side of the orifice, the orifice being so arranged that the

flow from it is directed downwards towards the hollow. This ensured effective dispersion of the water issuing from the orifice and resulted in relatively quite and free from reverse flow conditions in the pool; this later condition is a great advantage as the fish have a tendency to align themselves against the flow and thus lie facing downstream in the pool if reverse flow were there. The scale of the model was one-sixteenth full size and the difference in water level between pools corresponded to one foot six inches. The fish used for the experiments was small Trout which swam from pool to pool without difficulty.

THE STEEP CHANNEL TYPE

This type can be sub-divided as below :

(1) Steep channels with simplified baffles (with or without pools)

(2) Steep channels with side baffles.

(3) Steep channels with upward deflexion of jet.

(1) This pass consists of steep channels usually with slope of 1 : 5 with resting pools between. Baffles protrude from the bed usually at slope of 45° and rise upto the Full Supply Level. They are of simple design and can be readily constructed as precast reinforced-concrete-slabs. Sharp edges are avoided and the surface finished off to a reasonable degree of smoothness. The whole of the energy is dissipated in the channel and the sole function of the pools is to give a resting place for the fish. The pools should have a minimum size of 10 feet long by 10 feet wide and 5 feet deep. This type does not provide for much variation of head water level.

The variation of head water level can be best provided for by a series of pools with submerged orifices, thus making a combination of pool type and channel type passes in which the channel serves the normal height of the pass and the pools provide for the variation of headwater level.

(2) Steep channels with side baffles.

This type is roughened only on the sides by baffles, and there is no limit to depth. Where there is a moderate variation of water level, say of the order of 6 feet, this type can be advantageously used.

This form of channel, designed by Belgian Denil, has the characteristic that the velocity remains nearly constant with a variable depth of flow.

(3) Steep channels with upward deflexion of the jet.

This type consists of a plain channel with vertical *diaphragms* at suitable intervals dividing it into a series of pools. The diaphragms commence at some height from the bed thus leaving an orifice whose width is the width of the channel and the height the one at which the diaphragms commence. In each pool is a cross baffle at the bed about 6 inches high with an upward slope which deflects the jet upwards. Though this type is described as a channel its characteristics are those of a flight of pools. Increase in the depth of water in the channel will not effect the quantity of flow, as this is determined by the size of the orifice and the fall between the pools.

Variation of water level :—If the headwater level rises with a corresponding rise in the tail water level, a deeper channel will be required, but the flow will remain the same. If the head water level rises while that of the tail water remains constant, then the length of the channel will have to be increased and the depth will be increased at the upper end and taper down gradually to the lower end.

This form of pass is, therefore, adaptable to a variation of water level to the same degree as a pool pass with submerged orifice.

FISH-LOCK

The fish-lock consists of a vertical tower with an upstream orifice connecting with the headwater above the dam and a downstream orifice connecting with the downstream approach channel. The water enters the upper orifice and flows out through the lower one. Provided that the head and tail water levels remain the same, the water level in the tower will be constant, and this level will depend upon the relative sizes of the orifices. Both of these must be submerged.

The fish attracted by the current in the approach channel, will swim up and enter the tower, but the velocity through the upper orifice will be too great for them to pass through. When a number of fish are gathered in the tower the lower orifice will be closed by a sluice, the water level in the tower will rise and, under the reducing head, the velocity, though the upper orifice will decrease until a point is reached at which it is low enough for the fish to pass through. The filling of the tower can be accelerated if a sluice is attached to the upper orifice which can be opened up as the head reduces.

The orifices can thus be made of equal size, the upper one being throttled down to regulate the flow and gradually opened up as the water level rises.

In a high tower there is the possibility of fish attempting to pass through the upper orifice against too great a head and becoming unnecessarily exhausted. It will, therefore, be preferable for this orifice to be protected by a suitable grid and other orifices provided at higher levels, which can be opened when they have become submerged, to enable the fish to pass through. An alter native to the grid, which is likely to be safer for the fish, is to allow the upstream orifice to discharge into a well covered by a grating on which round pebbles are placed. The water will enter the pool by filtering upward through the pebbles.

Variation of water level :—If the headwater level varies, accompanied by a corresponding variation of tail water level, then the relative sizes of the two orifices will remain the same. If, however, the head water level rises while that of the tail water remain constant, then the upper orifice will require to be further throttled down. Provided that the two orifices are submerged below the minimum tail water level, then the two sluices will give complete control of the lock for any variation of water level.

The fish lock is well adapted to a high pass, with or without a large variation of water level, and under such conditions it is economical. Its drawback is that it requires the constant attendance of an operator when the fish are using it.

The Fisheries Division, Scotish Department have developed a fish-way of tower *cum* pool type of rather unusual construction. In this the pools rise spirally inside a concrete tower standing within the reservoir. To allow for the varying level of the reservoir, each of the upper pools is provided with a sluice controlled orifice giving direct access to the reservoir. The lower end of each of these fish-ways runs through a tunnel at the base of the dam to the pool where the fish congregate on the downstream side.

FISH-LIFT

A fish-lift or elevator has been used in America at high dams. The fish swim into a chamber through which water is running. The inlet and outlet are then closed and the chamber or tank is lifted bodily to the upper level opened for the fish to pass out. A lift is useful at a high dam where the rise is too great for any ordinary pass or even for a lock.

From a study of literature in connection with the fish-ways available in this office *we feel that no single design can suit all sites and varieties of fish met with, and that each project site needs a separate thorough study and investigation.*

The investigation on the subject may be divided into the following three parts ;

(*a*) a study of the habits of various kinds of fish,

(*b*) the capacity and strength of various kinds of fish to progress through different velocities and falls etc.,

(*c*) the design of the fish-pass to obtain conditions required *vide* (*a*) and (*b*) above.

The investigations under item (*a*) are purely a matter for the zoologists. In the foreign countries such investigations have been carried out for Salmon, Trout and Eel, the common varieties of fish met with in those countries. In India, however, mostly Hilsaw fish is found and it is necessary to carry out the investigations with regard to the habits and bionomics of this fish and other varieties of fish of economic value found in our country. *In fact it is most essential that a fish survey of the river for any new project should procede any attempt to design a suitable fish pass for the same.* This, among others, should comprise investigations of :—

(*i*) the type of fish met with,

(*ii*) their habits and bionomics, the period of ascent and descent and

(*iii*) situation of their spawning grounds.

It is gratifying to note that such research has been undertaken in Bengal although this will prove useful for that part of the country alone.

As regards investigations under item (*b*) a deeper insight into the problem of effort required by a fish is absolutely necessary and the three main phases of this can be defined as follows ;

(1) a study of the hydraulic properties of various fish-ways by measurement and observations on small models as well as on the full scale fish-way with appreciation of general laws of fluid mechanics,

(2) a study of the relations of fish effort to the properties of flow,

(3) determination of the limit of effort of each different species.

The success and correct design of the fish-pass as mentioned in item (*c*) *page 9 will depend upon the degree of correctness of the data obtained under items* (*a*) *and* (*b*) *with all their implications as mentioned above ; and this will be purely a matter for engineers.*

Research on the mechanics of fish mevements and design of fish-passes combining both biological and hydrodynomical problems is being carried out in the United Kingdom at Cambridge by Professor Grey. It would be advantageous if a suitable student engineer or an official is sent there to study the methods and the apparatuses required in this connection, who will on return be in a position to conduct similar research and experimentation for the various projects in this country and thus help in evolving the most suitable design for any particular site.

It will also be advisable if the Engineer who is deputed abroad is supplied with all the information regarding the working of the existing fish-passes on dams and weirs in India. This should include (1) type of fish pass (preferably a complete plan), species of fish using it, its working and further remarks and suggestion for improvements, if any, as a result of experience gained by the local officers.

In some cases after a result of fish survey we may even find that there is no need to construct a fish pass at all. Such would be the cases where the commercial value of a particular type of fish destroyed be not of much significance or where artificial spawning and hatching grounds for the fish could be introduced downstream of the obstruction. This has been done in the United States of America, on the Columbia River below the Grand Coulee Dam. The construction of this dam threatened the river fisheries with considerable losses because of the habits of the Salmon. The height of the dam, the irrigation pumping, and the long stretch of impounded water formed an unsurpassable grid obstacle to migration to the upper reaches of the river what to mention of the impossibilities of even getting the young fish back downstream. Induced migration to other branches of the river was, therefore, resorted to. The finding of new

spawning grounds, its planning and operation of the hatcheries is described by Dirk A-Dedle in the article " Preserving Columbia River Salmon Fisheries " in the April 1943 issue of the Civil Engineering Magazine published by the American Society of Civil Engineers.

CONCLUSIONS

(*a*) No useful purpose will be served at this stage by sending an officer to study just the fish-passes built in the foreign countries because ;

(1) it is essential that a fish survey of the river for any new project should precede any attempts to design a suitable fish pass,

(2) no single design can suit all sites and varieties of fish met with, and that each project site needs a separate thorough study and investigation,

(3) description of most of the passes built is available in the Board's Library and can be easily referred to by any body interested therein.

(*b*) To enable us to be in a position to find out the nautical properties of a fish, study of the apparatuses used for carrying out such experiments on scientific basis is absolutely necessary and for this somebody may be deputed to go and work in U. K. at Cambridge under Professor Grey.

DISCUSSION BY THE BOARD

THE SECRETARY said that there was no discussion on the subject at the Research Committee Meeting. Regarding the recommendations of the I. C. A. R. on the necessity of foreign study of fish passes and the scientific subjects related thereto, the Board last year decided that a note on the subject should be prepared in the Board's office. A note had been prepared by Shri P. R. Ahuja, Deputy Secretary of the Central Board of Irrigation and had been supplied to the members. The conclusions were given above and the Board might decide the action to be taken.

MR. R. R. HANDA was of opinion that no useful purpose would be served by sending an officer abroad to study this problem.

DIWAN BAHADUR N. GOVINDARAJA AYYANGAR said that on the Tungabhadra Project fish passes were required to be provided by the Fisheries Department but they had not collected any data about the life history of the fish there. So long as this data were not available they could not know anything about the characteristics of the fish. He thought that as much data as possible and statitics of fish habits should be collected.

RAI BHADUR S. K. GUHA wished that people study fish problems here first. The provincial Governments also be asked to consult the Fishery Department.

DR. N. K. BOSE said that he had seen how this work was done in America and London. Fish ladders were meant for the fish. There was much of hydraulics in it. The fish had got certain instincts as men have. They did not like certain things such as turbulence, falling water *etc.* All these might be studied also in a laboratory.

RAI BAHADUR C. L. HANDA stated that so far as high dams were concerned they were not provided with fish passes. The problems, therefore, did resolve themselves in the low barrages only. Punjab had already constructed certain passes in consultation with their Fishery Departments. The fish passes provided at Bonner Villi Dam were also good enough to be adopted. He was of the opinion that it was a problem which was manageable by the experience and talents in this country.

It was decided that it was not necessary to send any one abroad for the study of fish passes.

(v) OTHER WORKS

PRELIMINARY NOTE

Recent Literature.

(1) Suleiman, Hamed Bey, B.Sc. (London), A. M. Inst. C. E. Under-Secrtary of State for Public Works and Leliavsky Bey, Serge, Ph. D., M. Inst. C.E.M.A.S.C.E., Director, Designing Service, Reservoirs and Nile Barrage.—Observations of the movements of the lock wall of Aswan Dam—International Commission on Large Dams, Third Congress, Stockholm, 1948, R3.

(2) Bourriot, R., Ingenieur, E.N.P.C., France—Rupture d'une voute dessai (Breakdown test of an arch)—International Commission on Large Dams, Third Congress, Stockholm, 1948. R40.

THE YEAR'S WORK

The following items were discussed at the 1948 Research Committee Meeting :

(1) Ramapadasagar Dam sand sluices.

(2) Ramapadasagar Dam shooting bucket and high level sand sluices

(1) RAMAPADASAGAR DAM—SAND SLUICES ([3])

ABSTRACT

The tentative design of the Ramapadasagar project provided for sand sluices 10 feet×20 feet with centres at±0 in the body of the spillway. It was proposed to have one such sluice under every fourth span of the spillway. The working of the sluice with different levels of sand bed upstream and downstream and under different conditions of spillway and sand sluice discharges was studied on part model of the dam. Gives results.

([3]) Irrigation Research Station Madras, 1947, pages 85—91.

HYDRAULIC PARTICULARS

A cross section of the spillway including the sand sluice is furnished in Figure 5 C.1. Relevant hydraulic particulrs are as follows :—

Crest level of spillway	+170
F.R.L. (*i.e.*, top of gates)	+198
M.W.L. downstream	+90
L.W.L downstream	+43
Bed level upstream..	+0 to + 40
Bed level downstream	+0 to —20
Centre of sand sluice	±0
Size of sand sluice	10 feet× 20 feet
Invert of roller bucket	+24·0
Radius of bucket	80 feet

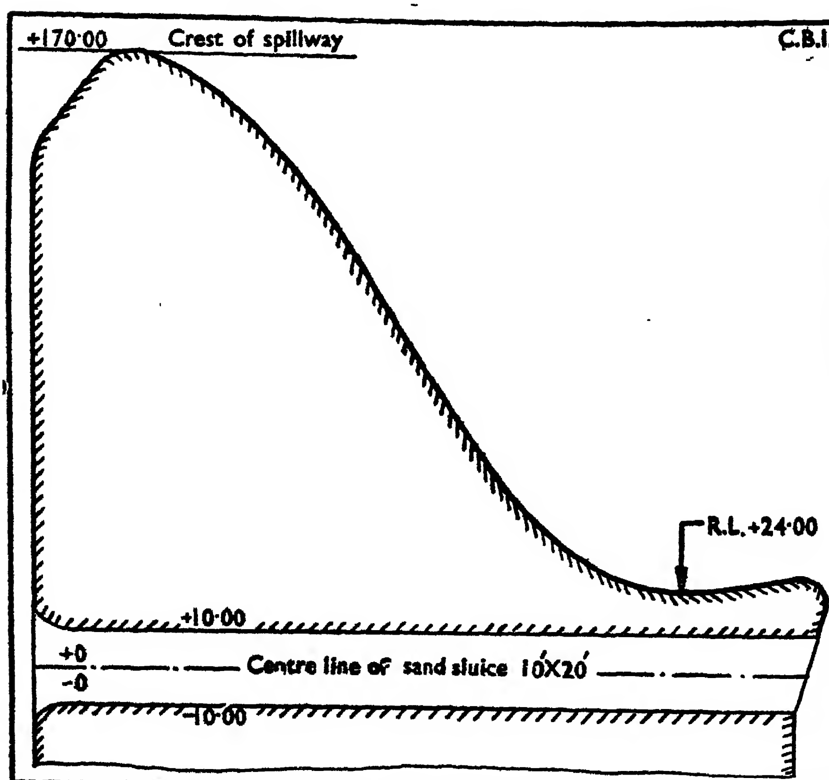

Figure 5 C.1 : *Ramapadasagar spillway dam showing sand sluice 10 feet×20 feet with 1/100 scale model*

THE MODEL

The model was a sectional one made to scale 1/100 and fitted up within the large glass sided flume of the Research Station which is 3 feet wide×3·5 fee

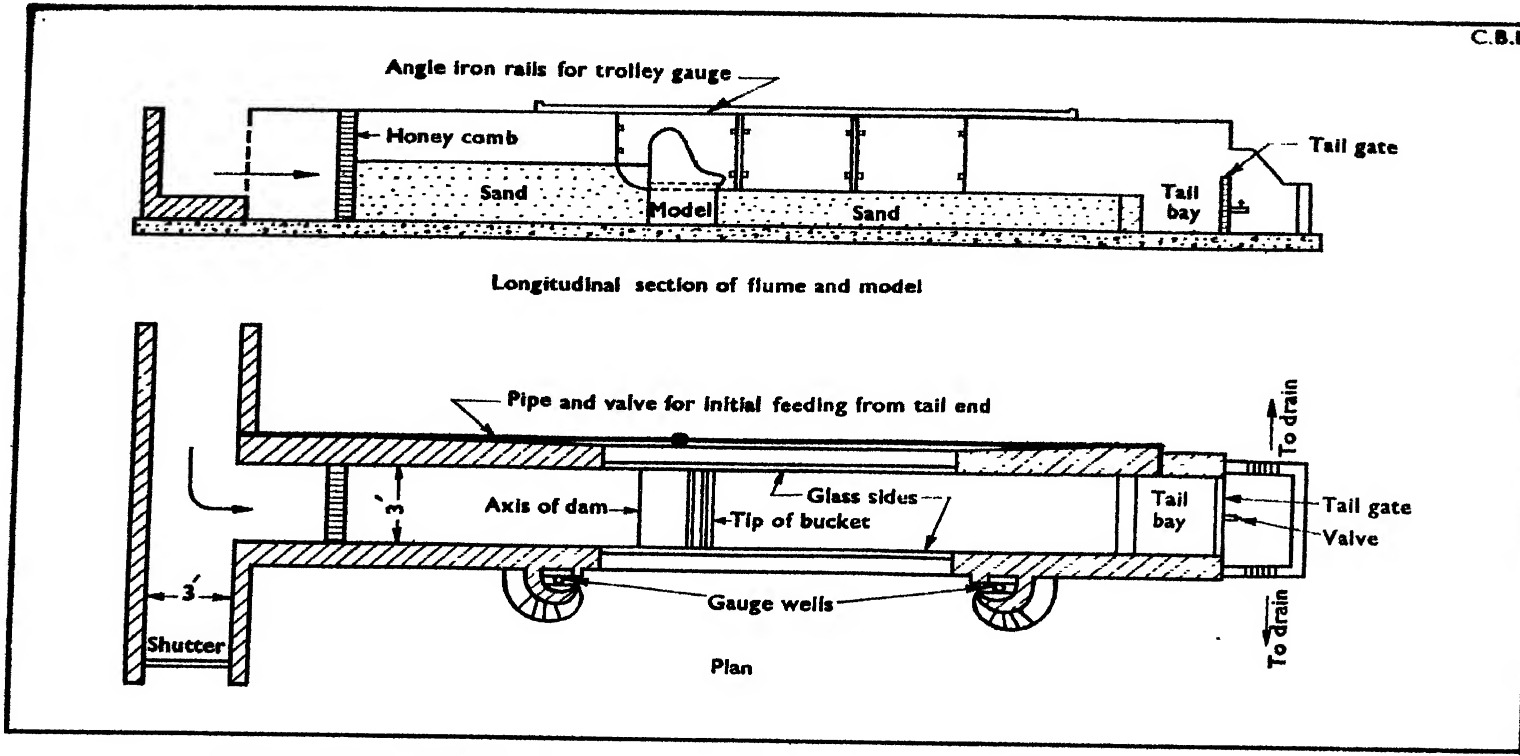

Figure 5C.2:- *Showing model setup of Ramapadasagar Spillway Sand Sluicies*

Scale 5 2·5 0 5 10 15 Feet

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deep and 35 feet long (*vide* Figure 5 C.2)—Water supply to the flume is drawn from the static tanks direct and controlled by sluice valves and overflow spills. Arrangements are provided for proper stilling of the inflow, for adjustment of tail water elevations, for feeding the model initially from the tail end and for measuring water depths, standing wave profiles and scour contours correct to a thousandth of a foot. The sand used for packing on both sides of the model was clean sieved sand passing through 14 and retained on 18 mesh per inch.

The model itself was made of well polished teakwood. It was provided with one sand sluice in the centre and another just near the glass face. The latter was operated for making qualitative observations on the nature of flow, *etc.*, and the former alone was operated in all experiments when quantitative data (such as scour contours) were taken. Excluding a large number of test runs and qualitative studies, a series of 14 experiments was made. These were made with different upstream and downstream water levels, with different initial bed levels upstream and downstream and under conditions of crest gate raised or lowered. The scheme adopted is given in Table 5C.1. In each experiment the period of run was 3 hours which was found sufficient to yield stabilised conditions upstream and downstream.

TABLE 5C.1

Ramapadasagar—Studies on Sand Sluices.

Experiment No.	Scheme of experiments				Remarks
	Upstream water level	Down-stream water level	Upstream bed	Down-stream bed	
1	2	3	4	5	6
1	+135·0	+43·0	0·0	—20·0	Sand sluice opened.
2	+198·0	+43·0	0·0	—20·0	Overflow.
3	+170·0	+43·0	0·0	—20·0	
4	+170·0	+43·0	+40·0	—20·0	
5	+198·0	+43·0	+40·0	—20·0	Overflowing the dam.
6	+198·0	+90·0	+40·0	—20·0	Do.
7	+170·0	+90·0	+40·0	—20·0	Do.
8	+198·0	+43·0	+40·0	—20·0	Drum gate in erect position.
9	+198·0	+90·0	+40·0	—20·0	Do.
10	+198·0	+43·0	0·0	—20·0	Do.
11	+198·0	+90·0	0·0	0·0	Do.
12	+198·0	+90·0	0·0	0·0	Do.
13	+198·0	+90·0	0·0	0.0	Do.
14	+198·0	+43·0	0·0	0·0	Do.

OBSERVATIONS AND DISCUSSIONS

In the qualitative studies the design of the sluice barrel was tested and found to work satisfactorily. When the sluice alone was discharging, the jet left the sluice in a dive and created scour on the bed downstream. The sand upstream was drawn gradually in a semi-cone and carried forward beyond the scour. There was some deposition downstream in some experiment due to lack of sufficient tractive force in the current. This may not happen in the prototype.

When the spillway was also discharging, the sand sluice jet was practically overpowered by the rollers of the spill. The scour occurred more on account of the spillway discharge than of the sluice discharge. Scour upstream sluice did form as it had formed with the sluice alone working, only it took more time to form. It was observed that :

(*a*) In any of the combinations tested there was no scour upstream with the bed initially moulded to $\pm$ 0. The sand sluices could not be effective in scouring out bed load below the sill level.

(*b*) With the bed upstream initially moulded to +40 the bed scoured to the shape of an inverted semi-cone with its apex at sand sluice sill and having the sides sloping to about 1 vertical to 2 horizontal. This occurred practically for all combinations of flow.

(*c*) With the sand sluice alone discharging a deep scour in the form of semi-ellipsoid formed downstream of the dam immediately opposite the sluice. The sand scoured out got deposited on the sides or got washed down below. The depth and size of scour varied with the head of discharge. These scours went down to about—50.

(*d*) With the spillway also discharging there was scouring but not of such concentration. The scoured bed went down to about—50 for extreme cases. With high tail water level the scour was much less.

CONCLUSION

The design of the shape and dimensions of the sluice were all right.

These sluices could not scour out sand deposited upstream except to a limited extent. The scouring action on deposited sand on the bed immediately upstream of a sulice was limited to an inverted semi-cone having its apex at the sill of the sluice, and sides sloping 1 vertical to 2 horizontal (nearly the angle of repose).

The effectiveness of the sluice should lie in disposing of the rolling and suspended silts as and when they approach the sluices; this would be governed by the discharge passing through the sluice.

The advisability of having a very larg· number of sluices was clearly indicated. The sills could be kept high enough to permit of being operated during first floods. This would also obviate the sluice mouth being blocked by deposition of silt in course of time.

(2) RAMAPADASAGAR DAM—SHOOTING BUCKET AND HIGH LEVEL SAND SLUICES (4)

ABSTRACT

The original designs of the Ramapadasagar Spillway Dam were made with the crest of spillway at + 170·0, F.R.L. at +198·0, bucket invert at +24·0 and sand sluice centres at +0. Various aspects of these designs have been tested on hydraulic models and reports describing the tests furnished. These studies and a few other considerations led to certain alterations in the design which was, therefore, proposed to have the following hydraulic particulars :—

Crest of the spillway	+180·0
F.R.L.	+200·0
Bucket invert	+74·0
Radius of bucket	50 feet
Tip of bucket	+81·0
Centre of sand sluice	+54·0
Size of sand sluice vent	10 feet × 20 feet
M.W.L. downstream	+90·0
L.W.L. downstream	+43·0
Bed Level downstream	+20·0

This design tested on a model to see the functioning of the bucket and sand sluice under various conditions of flow and the development of scours downstream. This report describes these studies.

MODEL

The model was a sectional one made to scale 1/100 and fitted up within a glass sided flume 3 feet × 3·5 feet × 35 feet having arrangements for water supply at constant head, for control of tail water levels and for measuring water surface profiles and scour contours correct to a thousandth of a foot.

The model was made of masonry and plastered smooth. The flume being 3 feet wide the spillway model represented a length of 300 feet in prototype, but no piers were introduced. One sand sluice vent was provided in the centre and another near one of the sides. The latter was operated only for visual observations and the former alone was operated whenever an experiment involving quantitative measurements was taken up.

Downstream of the model sieved sand passing through 14 and retained on 18 mesh per inch was packed to a depth of 13 inches with its top corresponding to plus 20.

The model was operated for various conditions of water levels upstream and downstream and for various combinations of flow over the spillway and through the sluice. In each case the period of run was 3 hours which was

(4) Irrigation Research Station, Madras, Annual Report, 1947, pages 91—96.

found to give stabilised scour downstream. Water surface elevations were traced during the runs and scour sections or contours were taken at the conclusion of the same.

EXPERIMENTS

Excluding a large number of trial runs, the following experiments were carried out :—

Experiment No. I.—Spillway alone discharging with maximum water level upstream and maximum water level downstream.

Experiment No. II.—Spillway alone discharging with maximum water level upstream and low water level downstream.

Experiment No. III.—Sand sluice alone discharging with maximum water level upstream and maximum water level downstream.

Experiment No. IV.—Sand sluice alone discharging with maximum water level upstream and low water level downstream.

Experiment No. V.—Spillway and sand sluice discharging both together with maximum water level upstream and maximum water level downstream.

Experiment No. VI.—Spillway and sand sluice discharging both together with maximum water level upstream and low water level downstream.

The water surface profiles, and sections and contours of scours obtained in these experiments were observed. A few salient features are discussed below.

FLOW CHARACTERISTICS

The spillway discharge issued beyond the tip of the bucket in a shooting jet for all conditions of flow. Its crest was at plus 98·4 which is less than 10 feet above the downstream maximum water level. The shooting jet dived down striking the river flow at a point 100 to 200 feet downstream of the bucket tip depending on the tail water level at an angle inclined about 15 degrees with the horizontal. A low standing wave was formed here creating secondary waves travelling downstream. There was not much disturbance, upstream, *i.e.*, between the point where the jet struck the water and the bucket tip.

The sand sluice was observed to run full when under the maximum head and the discharge issued beyond the sluice in the form of a gravity parabola.

When the sand sluice was worked along with the spillway overflow there was no bad effect on the flow characteristics.

SCOUR

Where the shooting jet struck the bed, scour started and developed upstream and downstream. The ground roller gradually built up a sand slope towards the bucket. With maximum water level in rear this slope was brought to the very tip of the bucket. At the tip there was no further movement and hence no likelihood of abrasion. With low water level in rear the sand slope could not rise above the water level and the downstream face of bucket was absolutely unaffected.

When the sand sluice was operated alone, it created a scour directly opposite. When the sluice, and spillway were both discharging, the scour was slightly different.

The extreme normal conditions were those tested in Experiments I, IV and V. In these experiments the deepest scours obtained were $-7 \cdot 4$, -30 and -15 respectively. Experiments II and VI should be considered as exceptionally rare conditions. Even here the worst scours were -56 and -40 respectively. On the whole the depth of scours might be treated as not too great. In previous experiments with 30 feet, overflow and plus $24 \cdot 0$ bucket the deepest scour with maximum water level upstream and downstream was -8.4. With maximum water level upstream and low water level downstream the scour went deeper than -100.

CONCLUSION

The design tested was found satisfactory. The crest of the shooting jet was at plus $98 \cdot 4$ and the deepest scour for the worst possible case was at -56.

DISCUSSION BY THE RESEARCH COMMITTEE

Mr. T. P. Kuttiammu introduced both the items and said that these and other studies on the Ramapadasagar dam design showed how the design was improved step by step by model experiments. At this dam the problem of exclusion of silt from the reservoir was of utmost importance. In the first design they provided, about forty sand sluices of 10 feet $\times$ 20 feet size with centres at $+0$ *i.e.*, near the bed. They found that the efficiency of the sluices lay in the quantity of muddy water disposed of by them. That called for a larger number of sluices. Eighty sluices were next proposed and finally this number was raised to one hundred and sixty, and this was considered necessary to dispose of all the normal floods.

When these sluices were kept near the bed there was the likelihood of their being blocked up by silt. The centres were first proposed to be at $+0$ next at $+54$, and finally at $+93$. In the final design the bucket and the sluices were all kept above M. F. L. in the river. Discharges over the spillway and through the sluices were to leave the bucket in shooting condition. As there would always be water to a depth of 40 to 90 feet downstream this design was found satisfactory.

It was decided that the subject should remain on the agenda.

DISCUSSION BY THE BOARD

The Secretary said that two items were discussed at the Research Committee Meeting (page 823). There was no resolution.

6C. Spillways

PRELIMINARY NOTE

This subject was introduced as a separate item for the first time in 1947. Previously discussion on the subject had taken place under "Dams".

The Board at its 1946 meeting accepted the recommendations of the Research Committee that Shri Ganesh Iyer's paper on "Spillway Siphons", be published as a Board publication after the author had made such additions and alterations, as he considered necessary. Shri Ganesh Iyer has not sent in the completed copy of the draft publication as yet.

There are four main types of siphon spillways. One is the overflow spillway, the second is the inclined tunnel spillway, the third may be called shaft spillway; this includes what the Americans call the "Glory Hole" spillways, and the fourth is the Ganesh Iyer's Volute Siphon.

The following are some of the features which require investigation:

(1) Overflow spillways

- (a) Slope of crest for maximum co-efficient of discharge
- (b) Slope of glacis and cistern for minimum erosion below the spillway.

(2) and (3) Inclined tunnel and shaft spillways

- (a) Mutual interference
- (b) Priming head
- (c) Co-efficient of discharge
- (d) Cavitation
- (e) Measures against erosion on the downstream side

(4) Ganesh Iyer Volute Siphon

- (a) Priming depth
- (b) Optimum slope of funnel
- (c) Shape, number of height and design of volutes
- (d) Determination of pressures on various points in the components of the siphon
- (e) Design of outlet and dissipation of energy below the outlet
- (f) Mathematical theory for the flow of water through the siphon.

The following items were discussed at the 1947 Research Committee Meeting :

Scour downstream of a five vented spillway dam for various combinations of shutter openings.

Pressure distribution on Ogee spillways Tungabhadra spillway dam.

Pressure distribution on the Ramapadasagar spillway dam.

Co-efficient of discharge for the Ramapadasagar spillway dam.

Stilling basin for overflow dams.

Roller bucket for the Ramapadasagar overflow dams.

Siphon spillways for Pahuj weir—A saddle type.

Preventing scour below Ganesh Iyer's Volute Siphon Spillways by mutual impact of jets issuing from adjacent siphon outlets.

Experiments on the effect of altering the shape and number of volutes in a volute siphon.

Experiments on dissipating the energy of flow from volute siphons.

Experiments on variations of co-efficient of discharge under varying degrees of submersion of siphon outlets.

Experiments on shafts spillways for tanks.

Model investigations regarding the design of Nagwa Dam.

Variation of co-efficient of discharge with reservoir levels for full openings of sluices in Krishnarajasagar Dam.

Rasul Hydel scheme spillway siphon.

Bhakra Dam spillways.

Recent Literature.

(1) Sailer R. and Davis B. G.—Unique caissons make spillway repairs possible at Grand Coulee—Civil Engineering, Vol. 16, No. 9, September 1947.

(2) Spillway energy dissipated on the face of a dam—Engineering News Record, Vol. 137, No. 24, December 12, 1946.

(3) Model studies of Spillway and Bucket for Centre Hill Dam, Caney Fork River, Tennessee—War Department, Corps of Engineers, Waterways Experiment Station, Technical Memo. No. 202-1, Vicksburg, Mississippi, August 15, 1946.

(4) Model study of Spillway Enid dam, Yacona river, Mississippi—War Department, Corps of Engineers, U.S. Waterways Experiment Station, Technical Memo. No. 2-223, Vicksburg, Mississippi, March 1947.

(5) Adams C. S.—Speedy repairs save Waco Dam spillway—Engineering News Record, Vol. 138, No. 12, March 20, 1947.

(6) Model study of the spillway and stilling basin, Harlan County Dam Republican River, Nebraska— U.S. Waterways Experiment Station, Vicksburg, Mississippi, Technical Memo. No. 2-236, September 1947.

(7) Use of models in design of spillways—Commonwealth Engineer, Vol. 34, No. 12, July 1, 1947.

(8) Liskovec Ladislav, Ing. The T. G. Masaryk National Hydrological Institute, Prague.—A special form of the spillway of a dam enabling the splitting of the nappe and the dissipation of its energy in the stilling basin.—Second meeting of the International Association for Hdyraulic Structures Research, Stockholm, 1948, Paper No. 6.

The Year's Work

The following items were discussed at the 1948 meeting of the Research Committee :

Battery of volute siphons.

Saddle siphons for Lalitpur reservoir.

Basic study of volute siphon.

Automatic gates on Tansa Waste Weir, Bombay—experiments with half size model.

Preventing scour below volute siphon spillways by fanning out jets with dispersers and ledges.

Scour downstream of a five-vented spillway dam.

Pressure distribution on Ogee spillways— Lower Bhavani Spillway Dam.

Co-efficient of discharge on Ogee spillways—Lower Bhavani Spillway Dam.

Co-efficient of discharge for Tungabhadra Dam.

Ramapadasagar spillway dam—effect of piers on co-efficient of discharge.

Spillway training walls for the Ramapadasagar Dam.

Stilling basin for Lower Bhavani Spillway Dam.

Tungabhadra Spillway Dam—three diamensional models.

High co-efficient weirs for irrigation tanks.

Siphon spillways formulae.

Further experiments on the dissipation of energy of flow from siphon outlets.

Design of volutes *etc.*, in siphons.

Experiments on the high head siphon at Shimsha.

Further experiments on siphons with various funnels and outlets.

Design of a high co-efficient weir and scour preventer for the Tunga Anicut.

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(1) BATTERY OF VOLUTE SIPHONS [1]

ABSTRACT

An idea of making a battery of volute siphons of Ganesh Iyer type was advanced by the late Chief Engineer, Mr. F. H. Hutchinson. He wanted this to be tried at Lalitpur reservoir under construction at the moment in Jhansi district. Though Gahesh Iyer Siphons are essentially suitable for higher heads, these batteries were agreed upon for this site as an experimental measure.

That the battery would materialise into a practical possibility was established on a large 1/12 scale model made at Bhadrabad according to the design. It was designed to operate at a head of 26 feet. It consisted of four volute siphons discharging through a tunnel of gradually expanding elliptical section. This yielded a co-efficient of discharge of 0·68 and priming depth 1·0 foot. There was some difficulty in perfect priming due to the outlet being a bit too wide. Similar

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(1) United Provinces Irrigation Research Station, Progress on Research Progress during 1947, pages 61—63.

study on a battery had also been carried out at Jhansi with almost similar results. The co-efficient of discharge in that case was slightly lower.

To perfect the design of such a battery basic study was necessary with regard to—

(i) the best shape and design of the tunnel,
(ii) optimum spacing of volute siphons,
(iii) optimum number of volute siphons.

(I) SHAPE OF THE TUNNEL

Three designs were tested on models to scale 1/24.

(*a*) Gradually expanding elliptical section giving at the junction of each siphon the area equivalent to the number of volute siphons upto that point, *vide* Figure 6C. 1.

(*b*) A rectangular section in which sudden expansions were provided to increase the cross-sectional area equivalent to the number of volute siphons upto that point *vide* Figure 6C.2.

(*c*) Same as (*b*) except that the sections were elliptical instead of rectangular, *vide* Figure 6C. 3.

The comparative model study of these tunnels yielded results with respect to priming depth and co-efficient of discharge *vide* Table 6C. 1. It is evident from the results that the priming depth for the rectangular tunnel was minimum in every case with different numbers of volute siphons. In the case of the two elliptical types of tunnel the priming was perfect only upto a combination of two volute siphons. With three and four siphons the priming needed a temporary obstruction at the outlet to make it run full bore. However, by extending the outlet by 33 per cent. perfect priming of the battery was obtained even with three and four volutes.

As regard co-efficient of discharge, the rectangular tunnel was the least efficient having the lowest co-efficient of discharge with different number of volute siphons in the battery. The gradually expanding elliptical section was the most efficient in every case; however, the co-efficients with rectangular tunnel were none too poor being only about 95 per cent. of the elliptical sections of similar design and 85 per cent. to 90 per cent. of the gradually expanding elliptical type of tunnel.

(II) OPTIMUM SPACING OF VOLUTE SIPHONS

Model studies on this aspect of the design were carried out by adding volute siphons one at a time upto four numbers for each type of tunnel *vide* Tables 6C. 2 to 6C. 4. The optimum spacing was found to be 48 feet for the maximum co-efficient of discharge. However, priming depth improved in every case with a nearer spacing.

(III) OPTIMUM NUMBER OF VOLUTE SIPHONS IN BATTERY

As will be seen from Table 6C. 1 the co-efficient of discharge deteriorates considerably when a battery increases from 1 to 2 or 2 to 3. But the extension from 3 to 4 does not show the same trend.

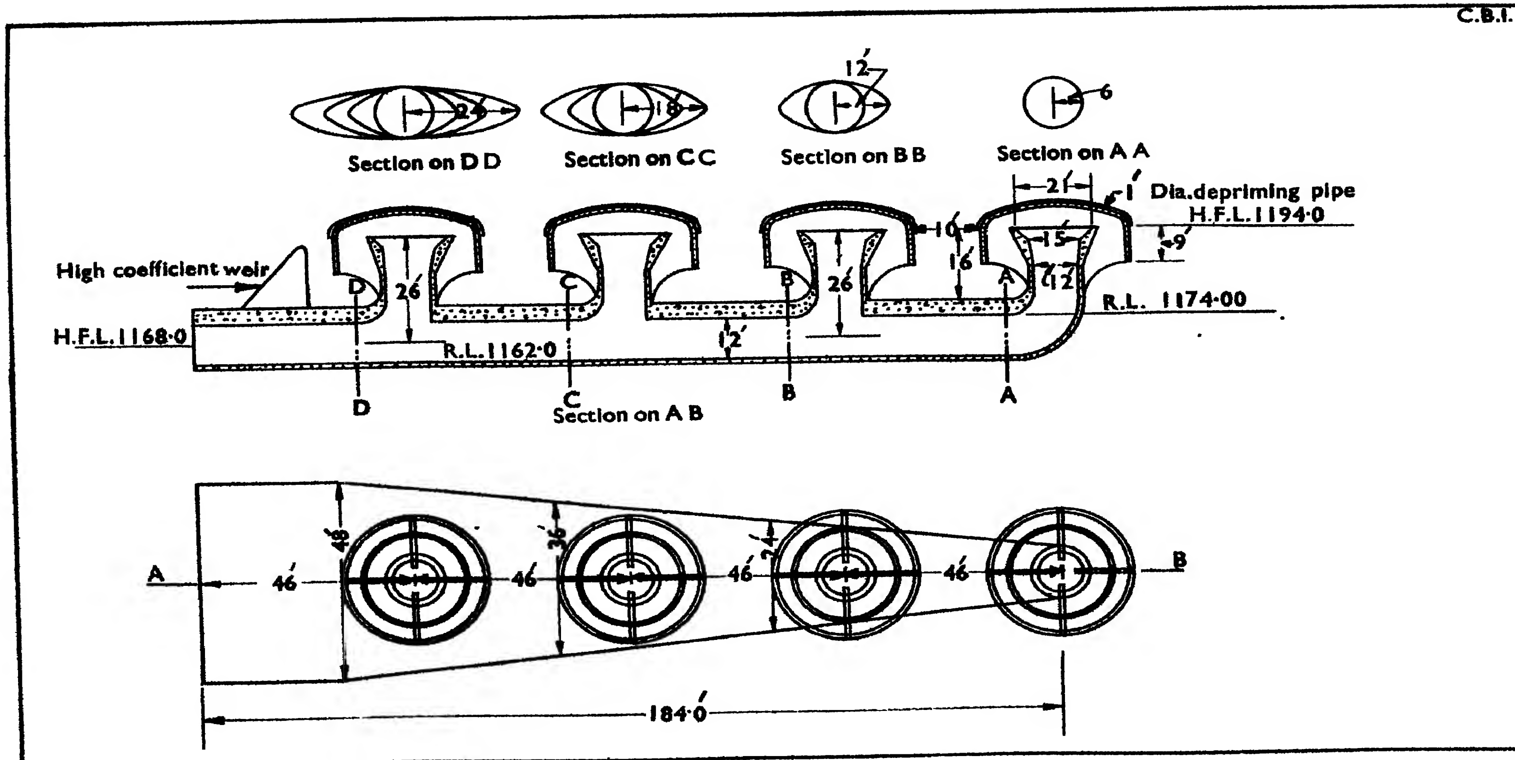

Figure 6C.1:-*Showing proposed design of a battery of four volute siphons with gradually expanding elliptical common tunnel (Lalitpur Reservoir)*

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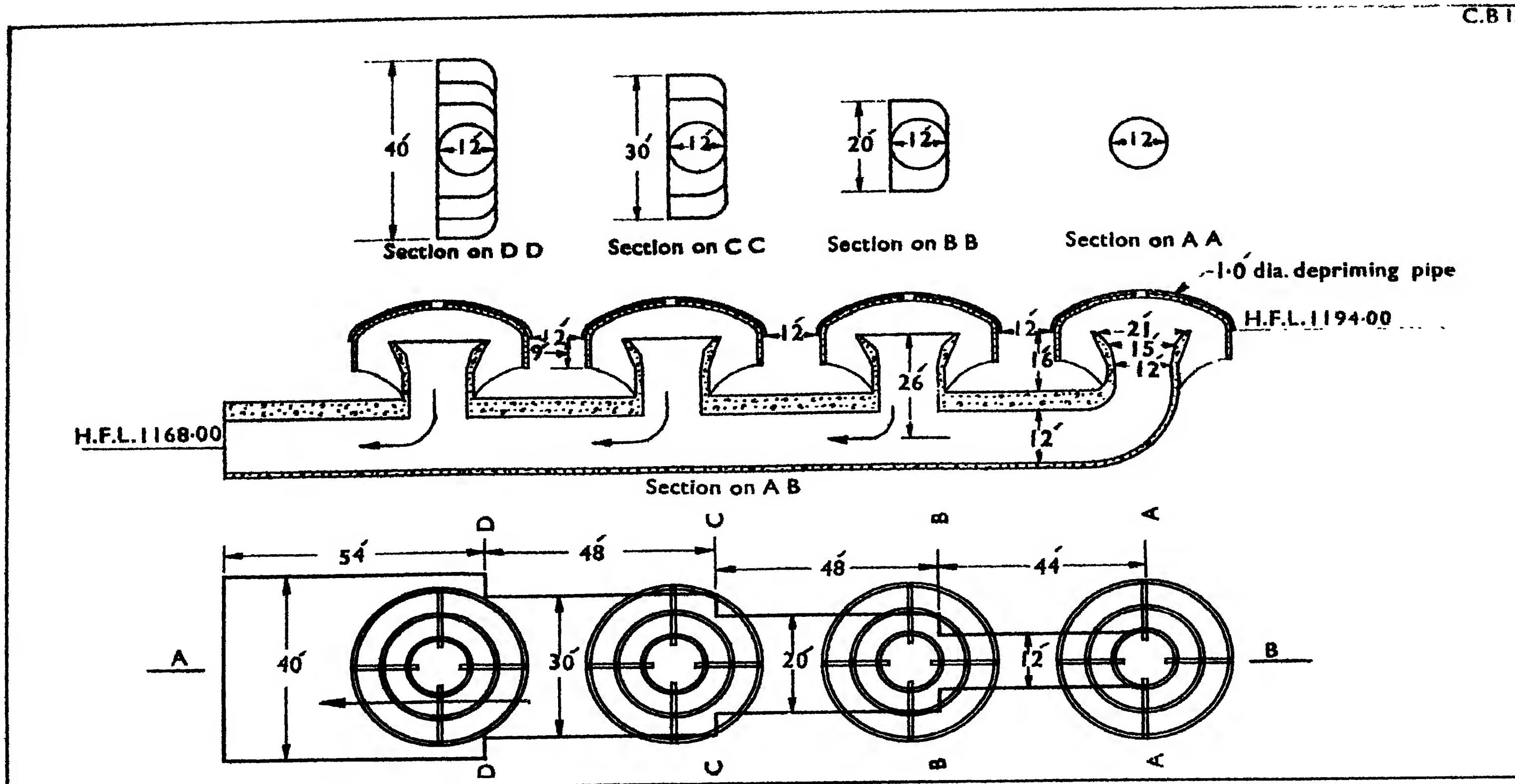

Figure 6C.2:- *Proposed design of a battery of four volute siphons with rectangular tunnel (Lalitpur Reservoir)*

Scale 50 25 0 50 100 Feet

Reg. No 2737 XDD (C)' 50 - 1,210 (P.Z.O.)

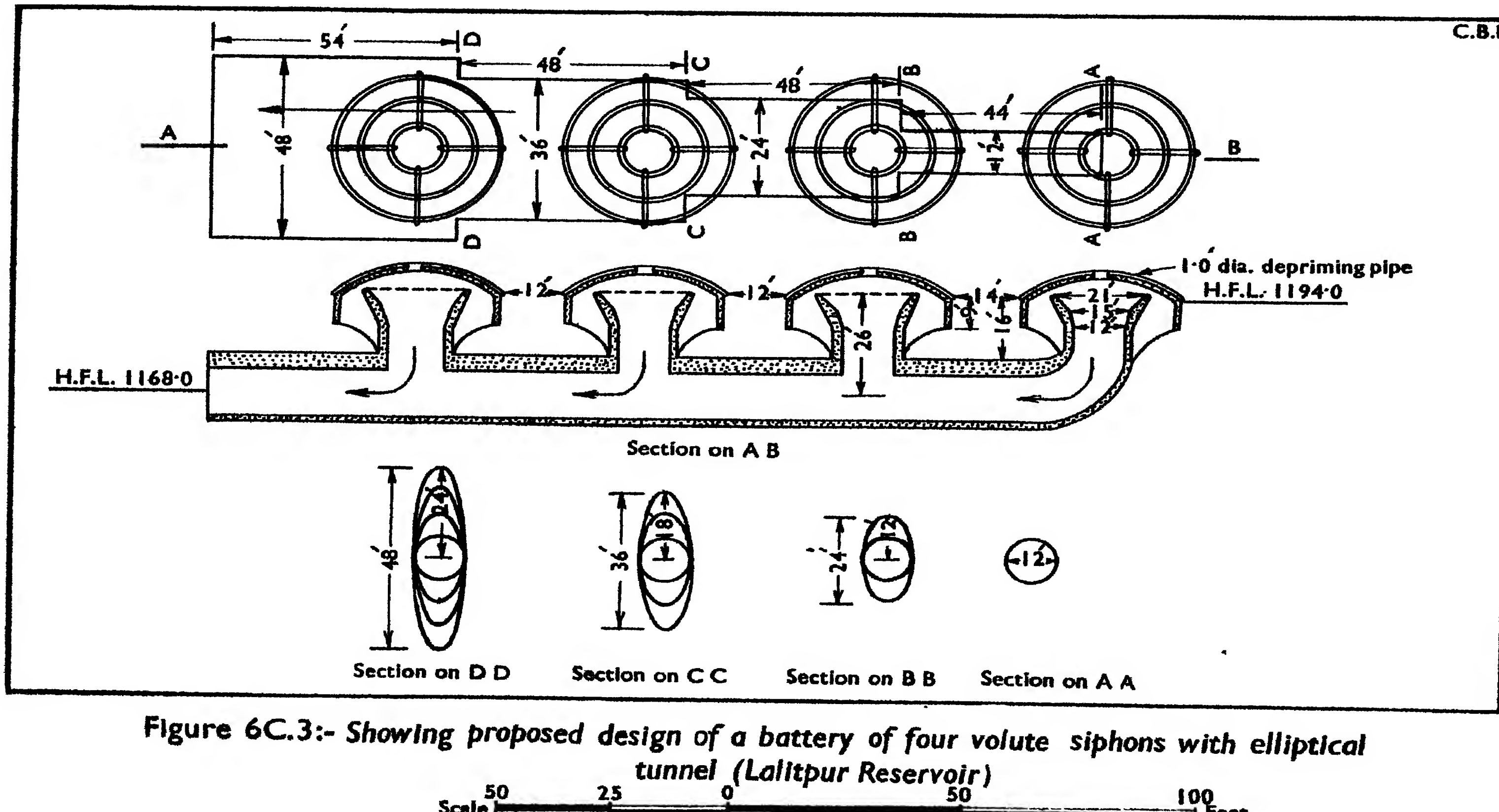

Figure 6C.3:- *Showing proposed design of a battery of four volute siphons with elliptical tunnel (Lalitpur Reservoir)*

Reg. No. 2737 XDD (C)' 50 - 1,210 (P.Z.O.).

CONCLUSION

The working of a battery is decidedly better in comparison to the single elliptical one which is likely to present abnormal structural difficulties in addition.

TABLE 6 C. 1

Serial No.	Number of volute siphons	Priming depth	Co-efficient of discharge
	Gradually expanding elliptical tunnel		
1	1	1·60	0·790
2	2	1·84	0·701
3	3	1·70	0·667
4	4	1·66	0·659
	Rectangular tunnel with sudden expansions		
1	1	1·60	0·790
2	2	1·63	0·648
3	3	1·48	0·561
4	4	1·43	0·552
	Elliptical tunnel with sudden expansions		
1	1	1·60	0·790
2	2	1·94	0·665
3	3	1·64	0·580
4	4	1·58	0·562

TABLE 6 C. 2

Serial No.	Spacing between siphons	Priming depth	Co-efficient of discharge
	Rectangular tunnel with sudden expansions		
1	40	1·501	0·610
2	42	1·488	0·620
3	44	1·512	0·632
4	46	1·536	0·642
5	48	1·582	0·646
6	50	1·632	0·648
7	52	1·704	0·650
	Elliptical tunnel with sudden expansions		
1	40	1·56	0·635
2	42	1·51	0·644
3	44	1·56	0·648
4	46	1·80	0·654
5	48	1·84	0·664
6	52	1·94	0·665
7	52	2·07	0·665

TABLE 6 C. 3

Serial No.	Spacing between siphons	Priming depth	Co-efficient of discharge
	Rectangular tunnel with sudden expansions		
1	40	1·37	0·530
2	42	1·39	0·534
3	44	1·42	0·558
4	46	1·44	0·559
5	48	1·48	0·561
6	50	1·52	0·561
	Elliptical tunnel with sudden expansions		
1	40	1·446	0·552
2	42	1·480	0·570
3	44	1·520	0·573
4	46	1·620	0·577
5	48	1·640	0·580
6	50	1·650	0·582

TABLE 6 C. 4

Serial No.	Spacing between siphons	Priming depth	Co-efficient of discharge
	Rectangular tunnel with sudden expansions		
1	40	1·360	0·545
2	42	1·370	0·546
3	44	1·410	0·548
4	46	1·410	0·550
5	48	1·430	0·552
6	50	1·460	0·541
	Elliptical tunnel with sudden expansions		
1	40	1·39	0·556
2	42	1·58	0·557
3	44	1·50	0·558
4	46	1·55	0·560
5	48	1·58	0·562
6	50	1·62	0·581

(2) SADDLE SIPHONS FOR LALITPUR RESERVOIR [2]

Since volute siphon batteries are likely to prove expensive owing to deep excavation in hard rock, it was subsequently decided to construct saddle siphons for this reservoir. They are required to pass 50,000 cusecs, with the available head of 20·0 feet.

On the basis of experiments conducted for Pahuj Siphon [3] a 15 feet × 10 feet saddle siphon was designed and tested in a glass model to scale 1/18 *vide* Figure 6C. 4. Crests of different radii were tried and the one which gave the optimum results as regards priming and co-efficient of discharge is shown in Figure 6C. 4. Table 6C. 5 gives the results of experiments conducted. The area of volute priming siphon has been kept 5 per cent. of the saddle siphon as suggested by Shree Ganesh Iyer in his paper presented to the 16th Annual Meeting of the Central Board of Irrigation.

TABLE 6C. 5

Serial No.	Type of crest	Priming depth in feet	Co-efficient of discharge
1	15 feet radius crest without lip	1·50	0·590
2	Do. with lip 1½ inch	0·72	0·550
3	8 feet radius crest without lip	2·75	0·704
4	Do. with lip 4½ inch	0·80	0·650
5	10 feet radius crest without lip	2·25	0·632
6	Do with lip 1½ inch..	0·72	0·613

([2]) United Provinces Irrigation Research Station, Report on Research Progress during 1947, pages 66—73.

([3]) U. P. Technical Memorandum No. 17 page 59.

These investigations led to the adoption of design as per case 6 above. The discharge through one saddle works out to be 3,300 cusecs, and that through priming siphon 350 cusecs. The design is yet to be finalised with regard to the size of priming and depriming pipes, and economical area of priming siphon for most efficient working. It was confirmed once again that although the area of priming siphon was increased to 10% of the saddle siphon, the priming depth continued to improve considerably whenever the nappe could strike the opposite wall inside. The priming was all the better when it struck the lowest point and smoothly slipped out adhering to the roof of the outlet. All this is mentioned to justify the provision of the 1½ inch lip which seems essential in this type of design when the conditions demand minimum priming depth as in this case, the full supply level having been kept the same as high flood level.

Figure 6C. 5 shows pressures inside the body of the siphon. Possibility of a little cavitation is indicated at the crest only.

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(3) BASIC STUDY OF VOLUTE SIPHON [3]

Basic work was conducted last year on Ganesh Iyer's volute siphon to determine its outlet length for maximum co-efficient of discharge and minimum priming depth. Further work was continued this year with regard to :—

(*i*) Effect of diameter of the covering hood on priming depth and co-efficient of discharge.

(*ii*) Effect of variation of working head on the priming depth and co-efficient of discharge.

(*iii*) Effect of variation of working head on the maximum negative pressures developed.

Drawing No. 25 of U. P. Technical Memorandum No. 17 shows the design experimented upon on a model, built 1/8 to scale.

(*i*) Hood in the original design of volute siphon of 3·0 feet in diameter, was 8 feet and this was gradually varied to 12·0 feet. The priming depth and co-efficient of discharge were observed under constant head. The results are tabulated *vide* Table 6C. 6.

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[3] United Provinces Irrigation Research Station, Report on Research Progress, during 1947, pages 68—73.

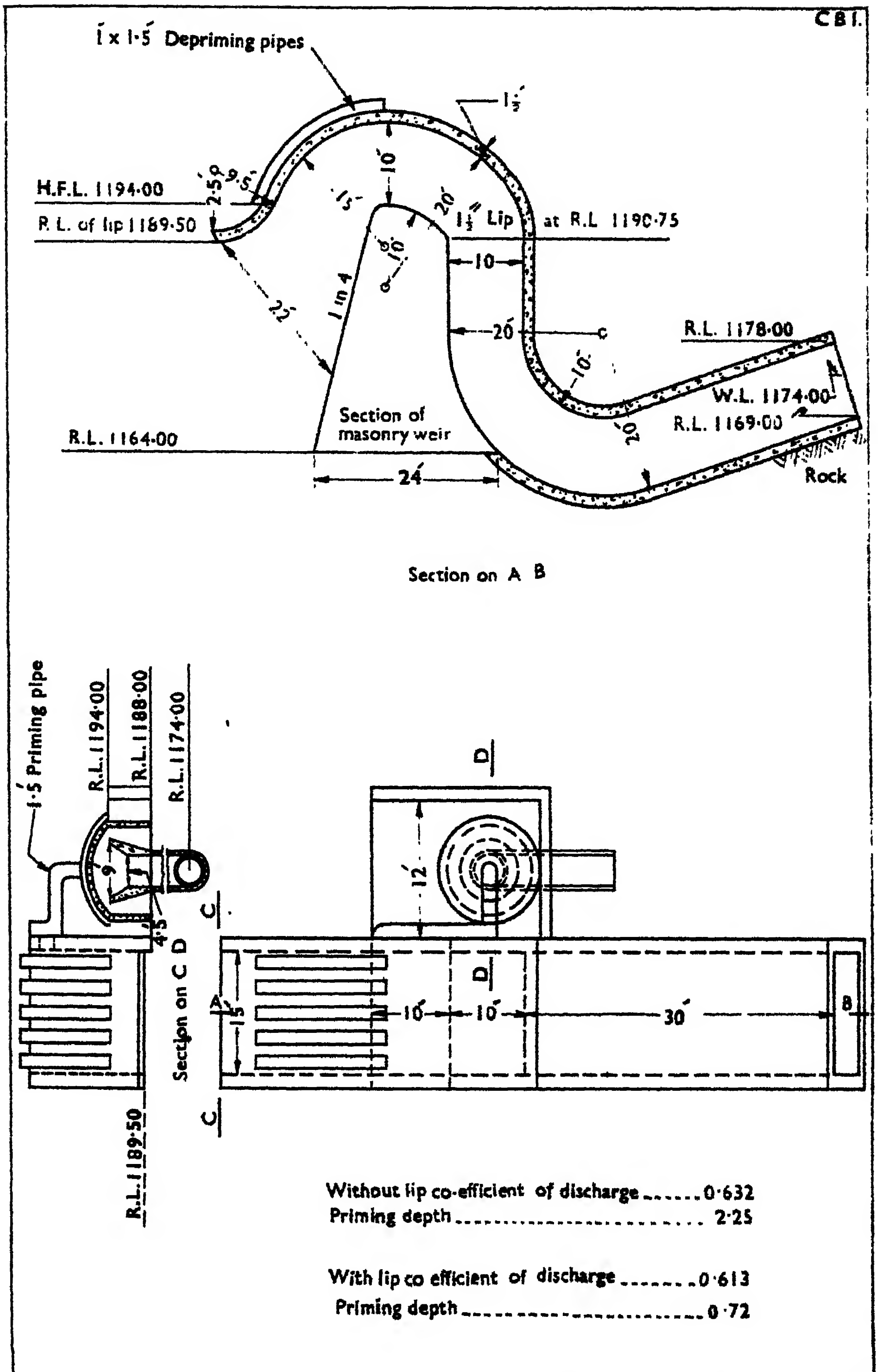

Figure 6C.4:- *Showing saddle type siphons for Lalitpur Reservoir*

Scale 20 10 0 20 40 Feet

Reg No. 2737 XDD (C)'50-1,210 (P.Z.O.).

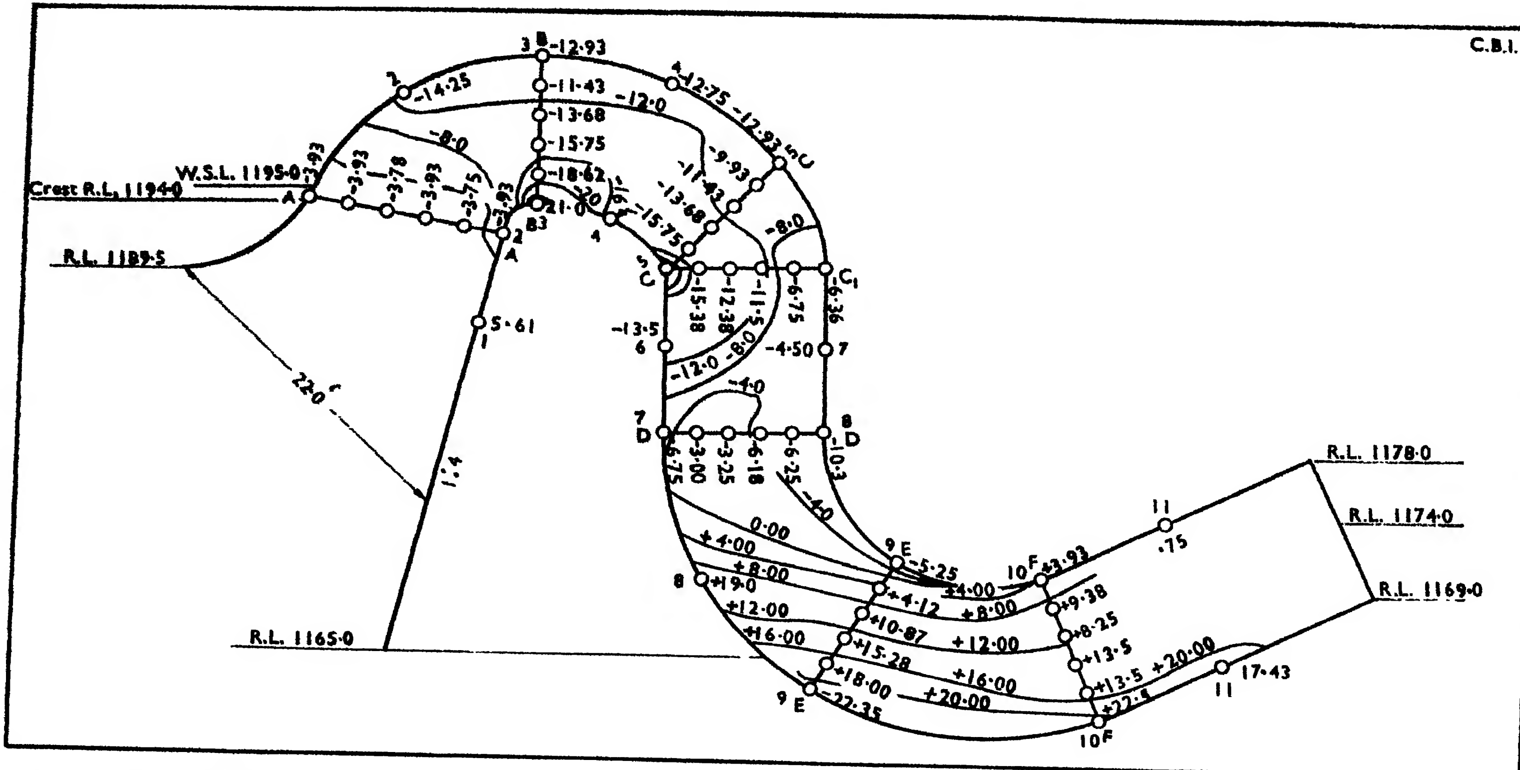

Figure 6C.5 :- *Showing pressure contours within siphon barrel drawn at 4·0' change of head, atmospheric pressure 0·00*

[Saddle type siphon for Lalitpur reservoir]

Reg. No 2737 XDD (C)' 50 - 1,210 (P.Z.O.)

It is clear that maximum co-efficient of discharge and minimum priming depth is indicated when diameter of covering head is 3·33 times the barrel diameter.

(*ii*) Effect of priming depth and co-efficient of discharge when the working head was gradually changed, but upstream water surface was maintained constant, was studied on the same model. Table 6 C. 7 shows the observations when the working head was varied from 10·0 feet to 22·50 feet.

The graph indicates that as the head increases both the priming depth and co-efficient of discharge reduce. Further experiments are desirable to confirm the above trend for higher heads.

(*iii*) Variation of maximum negative pressure, developed as the working head was changed, was also studied on the same model. Table 6 C. 8 shows the above variation. The corresponding maximum negative pressure is lower for lower heads and higer for the higher heads.

The Ganesh Iyer Siphon seems more suited for high heads than the low ones as, only in the former case, the maximum desirable limit of negative pressure 24 feet can be possibly attained.

TABLE 6C. 6

Serial No.	Diameter of covering Hood in feet	Hood diameter / Barrel diameter	Priming depth	Priming depth / Barrel diameter	Co-efficient of discharge	Remarks
1	8	2·66	0·400	0·133	0·706	Diameter of the barrel of volute siphon is 3·0 feet.
2	9	3·00	0·320	0·106	0·706	
3	10	3·33	0·248	0·083	0·708	
4	11	3·66	0·288	0·096	0·707	
5	12	4·00	0·318	0·106	0·706	

TABLE 6 C. 7

Serial No.	Designed head	Designed head / Barrel diameter	Priming depth	Priming depth / Barrel diameter	Co-efficient of discharge	Remarks
1	10·00	3·34	0·432	0·144	0·721	Diameter of the barrel of volute siphon is 3·0 feet.
2	12·50	4·17	0·400	0·133	0·712	
3	15·00	5·00	0·360	0·120	0·701	
4	17·50	5·83	0·320	0·107	0·692	
5	20·00	6·66	0·296	0·099	0·686	
6	22·50	7·50	0·288	0·096	0·682	

TABLE 6 C. 8

Serial No.	Working head	Maximum negative pressure head developed	Remarks
1	10·00	—11·68	Upstream water surface over the crest of the siphon was maintained constant.
2	12·50	—12·24	
3	15·00	—13·04	
4	17·50	—14·64	
5	20·0	—17·52	
6	22·5	—19·04	

(4) AUTOMATIC GATES ON TANSA-WASTE WEIR, BOMBAY, EXPERIMENTS WITH HALF SIZE MODEL (4)

ABSTRACT

In order to provide partial relief against the acute water-shortage facing the Bombay city, it was decided to increase the storage capacity of the Tansa Lake by raising the retention level—and, therefore, the crest level of the waste weir— from R. L. 420·00 to R. L. 422·00 T.H.D. (Town Hall Datum) by installing automatic gates on the weir wall. Experiments with 1/8 and 1/10 scale geometrically-similar models were conducted in 1945 at the Poona Station to collect data necessary for the design of the automatic gates (5). On the basis of these data the automatic gates were tentatively designed. Experiments contemplated with a full scale gate not being feasible at the Station, experiments with half size geometrically-similar model of the weir and the gate were made in the 8 feet wide flume. The length of the gate was, however, equivalent to 16 feet only as against 50 feet in the prototype. Results are given.

THE MODEL

The height of the Tansa weir is 130 feet. In the model only the top 20 feet were reproduced. As the ratio of height of weir to depth of water in the model was not less than 10, the velocity of approach was negligible.

The 8 feet wide flume had direct connection with the Lake, thus allowing any desired discharge to be drawn. Discharges up to 65 cusecs were run in several tests. The discharge passing over the gate was measured over a five feet wide standing wave flume, 50 feet below the gate. To even out the distribution of water and damp down the turbulence, elaborate arrangements consisting of a combination of baffles, grids and cross-wall were provided with marked success.

Accurate recording of water levels was done in a gauge chamber, 11½ feet upstream of the gate.

Air vents were provided in the side walls to be open to atmosphere through vertical shafts connected with the vents.

REQUIREMENTS AND PROGRAMME OF EXPERIMENTS

The gate was designed to operate as soon as the water level rose/fell just above/below the *retention* level (R. L. 422·00) and, in its final position, to lie at an angle of about 4° with the horizontal at the highest *permissible* water level of R. L. 422·25. Actually, the dam section is designed to be stable at the *maximum* water level of R. L. 422·50; but a margin of 0·25 foot between the *permissible* and *maximum* water levels was provided in the interests of safety.

(4) Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report Technical, 1947, pages 153—164.

(5) Indian Waterways Experiment Station, Poona, Annual Report, Technical, 1945, Item 11

The gate was required to escape the flood discharge equal to about 6·45 ≡ 18·25 cusecs at the *permissible* water level 422·25 T. H. D.

An important requirement was that the gate should be sufficiently sensitive to close completely at the *retention* level so that there would be no loss of storage capacity below retention level.

The following programme of experiments was fixed to test whether the gate behaves correctly according to design :—

Experiment No. 1.—To ascertain the exact water levels at which—

(*a*) the gate starts opening;
(*b*) the gate is fully down;
(*c*) the gate starts returning; and
(*d*) the gate is completely shut.

Experiment No. 2.—To ascertain the discharge passing over the gate when the lake level in the prototype is at—

(1) R.L. 422·125;
(2) R.L. 422·250;
(3) R.L. 422·375; and
(4) R.L. 422·500.

These experiments were to be done *with* and *without* the anti-vibrations.

Experiment No. 3.—To determine and tabulate the water pressure curve on the deck plating at three different lines on the gate and for the following inclinations of the gates with the horizontal :—

(*a*) 52°—30′
(*b*) 50°—00′
(*c*) 45°—00′
(*d*) 40°—00′
(*e*) 35°—00′
(*f*) 30°—00′
(*g*) 25°—00′
(*h*) 20°—00′
(*i*) 15°—00′
(*j*) 10°—00′
(*k*) 5°—00′
(*l*) 0°—00′

Experiment No. 4—To study the effect on the discharge and operational behaviour of the gate of different weir profiles with a view to obtain *optimum* conditions in respect of discharge and aeration.

From the model experiments it was concluded that;

(*i*) The final design now adopted—shown in Figure 6C. 6 will pass a minimum of 4·7 ≡ 13·3 cusecs discharge in the *rising* flood under *fully aerated* conditions, against 6·45 ≡ 18·24 cusecs required.

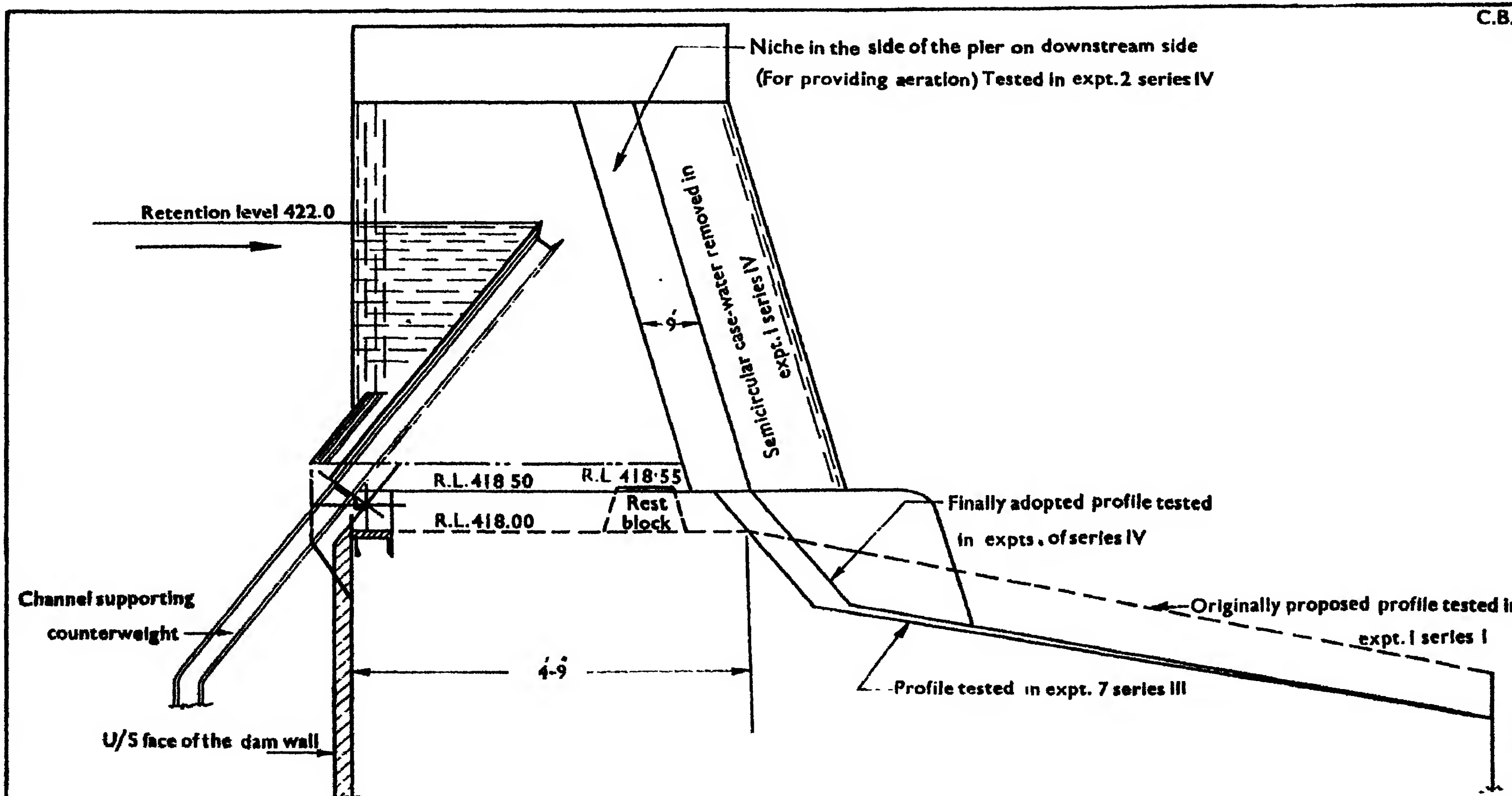

Figure 6C.6:- *Showing profiles of the Tansa waste weir tested in some of the experiments & also the final profile accepted for adoption on site*

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Under the *falling* stage conditions, with *aerated* nappe, the discharge will be more than under rising stage conditions; but still less than the necessary discharge at W. L. 422·25; while with *unaerated* nappe the discharge will be in excess of the minimum necessary.

To pass the required discharge of 18·24 cusecs on *rising* flood the upstream water level has to be higher, say, R. L. 422·40 T. H. D.

(*ii*) In the profile of the final design, the resultant falls outside the middle third at section R. L. 365·00 with W. L. 422·25 by 0·3 foot *i.e.* nearly by the same amount as in the original profile accepted by the Bombay Municipality.

(*iii*) It is not possible to say with certainty that the device which gives aeration in the model will give equally good aeration in the prototype (for reasons already mentioned), or *vice versa*.

(*iv*) It has been observed that aeration is important for sensitivity *i.e.* the smooth working of the gates; as, *inter alia*, the counterweight otherwise falls rather suddenly against the brackets.

(*v*) On the other hand, unaerated conditions give a somewhat *higher* discharge.

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(5) PREVENTING SCOUR BELOW VOLUTE SIPHON SPILLWAYS BY FANNING OUT JETS WITH DISPERSERS AND LEDGES [6]

ABSTRACT

Experiments were carried out last year in which attempts were made to prevent scour below Volute Siphons by making the high velocity jets hit each other at an angle [7]. This method of dispersion was not found satisfactory as the maximum depth of scour under these conditions was more than obtained with siphon jets issuing straight or acting singly.

In experiments carried out this year, jet dispersers, *with* and *without* a tilting ledge, were fixed at the siphon outlets so that the jets were spread out evenly, and at the same time deflected upwards at various angles, before impinging. This resulted in reducing the intensity of the jet and in lesser scours downstream. As anticipated, there was a small reduction in the coefficient of discharge.

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[6] Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report Technical 1947, pages 165-166.

[7] Indian Waterways Experiment Station, Poona, Annual Report, Technical, 1946, page 91.

MODEL AND PROCEDURE OF EXPERIMENTS

Two volute siphon models, with operating head of 2·064 feet, were used. The diameter of the vertical shaft was four inches. The bed downstream of the siphons was laid to a depth of two feet in incoherent sand of mean diameter 0·4 m.m. In all the experiments, a constant water level was maintained upstream of the siphons and also over the sand bed downstream of the siphons. The siphons were run for a period of 10 hours which was found quite sufficient to give comparative results with the different dispersers tested though not sufficient to give final stable scour-holes; because 90 to 95 per cent. of the ultimate scour depth is reached within 5 to 10 per cent. of the run required for the final scour and the rate of scour thereafter is very slow.

Figure 6 C. 7.—*Photograph showing jets in siphons with disperser, and a ledge at an angle of 45°*

Figure 6 C. 8 : *Photograph showing jets in siphons with disperser, and a ledge at an angle of 20°.*

Figure 6 C. 9 : *Photograph showing downstream bed after ten hours run with disperser, and a ledge at an angle of 45°.*

Figure 6 C. 10: *Photograph showing siphons with dispersers, side expansion 4:10 (with ledge not in action).*

These experiments have shown that the arrangements as tried in the above experiments would substantially reduce the depth of scour and would make costly protective works on river beds unnecessary. Figures 6 C. 7, 6 C. 8, 6 C. 9 and 6 C. 10 showing the fanning out of jets obtained with dispersers and a ledge.

(6) SCOUR DOWNSTREAM OF A FIVE-VENTED SPILLWAY DAM [8]

A set of three experiments were carried out to determine the relative advantage of having rectangular or trapezoidal staggered blocks at the end of the apron.

Scours obtained in the three experiments showed that scours were definitely reduced by the use of staggered blocks. The difference in scours obtained for the two types of blocks used was, however, insignificant. Though generalisation from this one experiment would not be correct, it could be inferred that a slight alteration in the shape and dimensions of staggered blocks would not affect their hydraulic efficiency and could be made if dictated by consideration of economy or structural stability.

In Chapter III-C, pages 29—41 of the annual report of the Research Station for 1946, a series of experiments conducted on the model of a five-vented spillway dam was reported. These experiments showed the nature and development of scours downstream of the dam for various combinations of shutter openings when the river downstream was level with the apron and confined between two masonry walls 6 feet apart in the model parallel to and symmetrical about the axis of the river which was normal to the dam. The studies reported indicated that, for the disposition of the river tested, namely, with a level bed and symmetrical margins, it was best to open all shutters partly and uniformly for allowing the desired discharge instead of a few of them fully; in case only a few shutters were to be opened, it was better to distribute them evenly along the full length of the regulator and to prefer a symmetrical arrangement of opening.

These studies were continued with the width of the river reduced to $3\frac{1}{2}$ feet measured parallel to the spillway and the side walls placed inclined 1 : 4. This represents a condition commonly met with in practice and the experiments could be expected to yield useful indications.

EXPERIMENTS

One experiment was carried out with all the five-vents open. All other experiments were made with two vents open in ten combinations, *viz.*, (1 and 2, 1 and 3, 1 and 4, 1 and 5, 2 and 3, 2 and 4, 2 and 5, 3 and 4, 3 and 5 and 4 and 5). Subsequently, a few experiments were carried out with side walls replaced by erodible soil banks. The scour contours obtained indicated that, in a situation similar to the one tested, the best method of regulation will be (*i*) to avoid concentration of flow towards the projecting (left-margin), (*ii*) to avoid as far as possible any concentration of flow on the other (right) margin as well, (*iii*) to prefer opening of shutters near about the centre slightly to the right and (*iv*) to distribute the openings in an even manner. The basic idea again is the elimination or mitigation of lateral eddies. In our earlier studies, the eddies were caused by unbalanced opening of shutters; in the present case, an additional factor has come in, namely, the obliquity of the margins.

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(8) Irrigation Research Station, Madras, Annual Report, 1947 pages 10—*21*.

ERODIBLE MARGINS

A few of the experiments were repeated after replacing the side walls with erodible banks made up of a soil containing about 50 per cent. sand and 50 per cent. clay and silt. The scours and erosions here developed into the margin but depths of scours got reduced. In other respects, the conclusions above stated were found to tally.

(7) PRESSURE DISTRIBUTION ON OGEE SPILLWAYS—LOWER BHAVANI SPILLWAY DAM (9)

INTRODUCTION

Two profiles, present and future, of the 15 feet overflow section of the Lower Bhavani Dam are shown super-imposed one over the other in Figure 6 C. 11. The final profile is based on the underside of the standard nappe and the present profile is inscribed within this suitably.

Both the profiles had to be checked with a view to locate the possible development of negative pressures on the masonry and to search for economies. Possibilities of the development of negative pressures were present in both designs : in the ultimate profile, due to the extremely economic section adopted ; and in the present profile, due to the transitions formed at the junctions of the several curves. The pressures were tested on 1/36 sectional models operated under M.F.L. conditions.

MODEL

A model of the spillway (final profile) to scale 1/36 was constructed in smoothly plastered masonry, fitting it up between the parallel walls of a masonry flume 25 feet × 2 feet × 5 feet. Piezometer points were introduced along the centre line of the profile. These points were connected by suitable tubings to a manometer set up outside the flume wall.

The flume took off from a stilling chamber to which water was supplied at constant head by means of inlet and overflow valves.

Before starting an observation the manometer tubes were all filled with water, air locks were sucked out and correct level in each tube corresponding to zero pressure at the concerned point was carefully marked. Water was then let in and when M. W. L. conditions were obtained and kept steady the rise or drop as the case may be of water level in each tube was measured. The study was repeated for confirmation.

The model was then altered and remodelled to the present profile and similar observations were taken.

(9) Irrigation Research Station, Madras, Annual Report, 1947, pages 46—48.

PRESSURE OBSERVED

The pressures were observed on the two models and the following features were noted :—

(*i*) The pressures everywhere were zero or positive ; there was no negative pressure anywhere.

(*ii*) Just below the crest the pressures were very nearly zero along the surface of the final profile. This indicated a highly efficient design. A co-efficient of discharge slightly exceeding 3·9 (in $Q=CLH^{\frac{3}{2}}$) could be expected.

(*iii*) The pressures were of the order of 1 to 3 feet of water in the same region for the present profile. The co-efficient of discharge would be about 3·8.

(*iv*) The pressures became zero in the region of + 880 for the present profile. This was due to the change in the curve in the region. That the pressures had not fallen below zero was an indication that the design was sound.

(*v*) On the whole both the designs were sound and economic. The results could be treated as of basic value applicable to similar designs adopting Creager's profile.

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(8) CO-EFFICIENT OF DISCHARGE ON OGEE SPILLWAYS—LOWER BHAVANI SPILLWAY DAM (10)

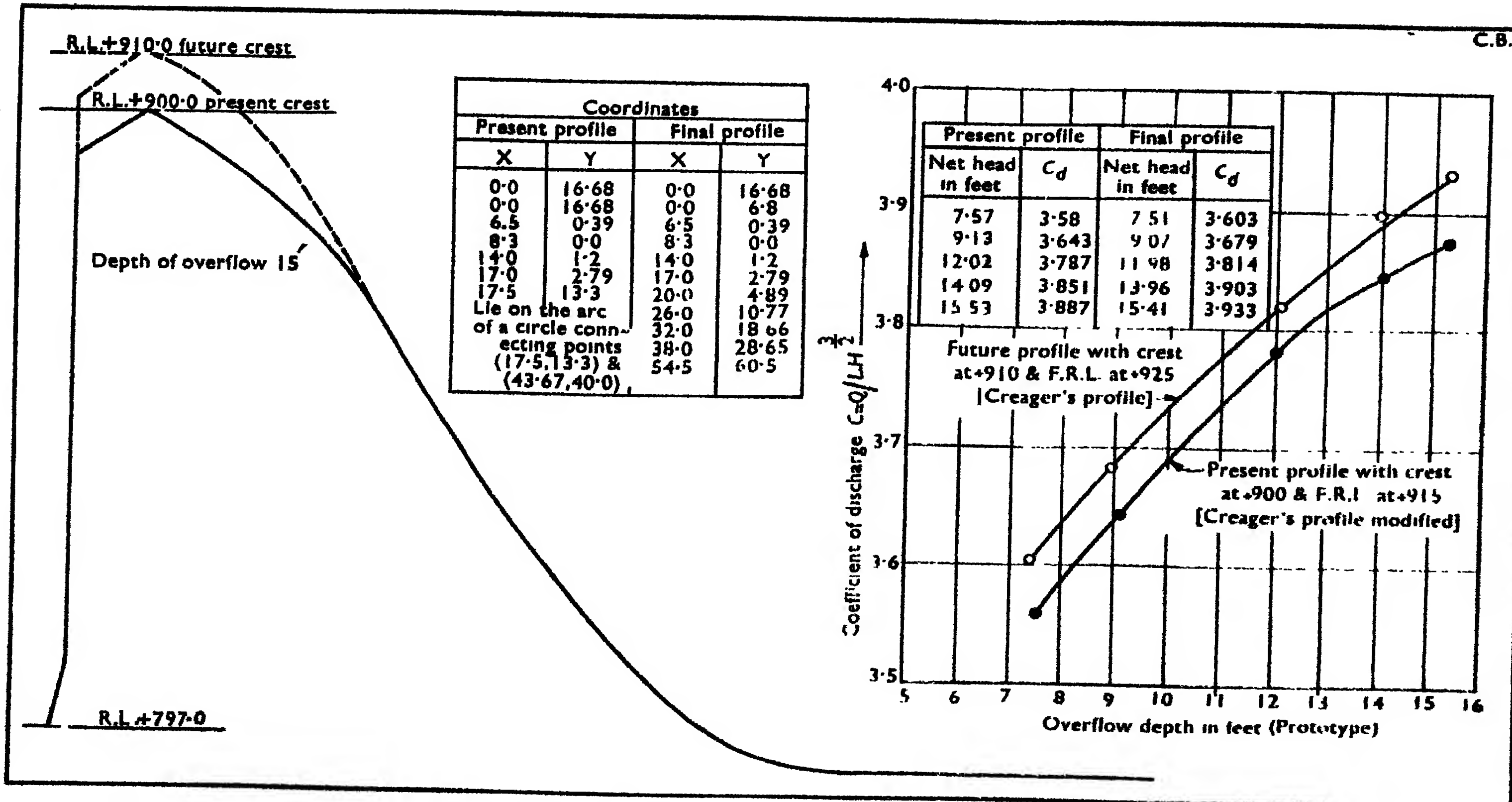
INTRODUCTION

Piezometer studies made on the two profiles present and future of the 15 feet overflow section of the Lower Bhavani Dam were described in item (7) above. The profiles being of a basic design it was considered desirable to obtain their co-efficients of discharge by model experiments. Several experiments were carried out in this connection of which only one set is described below.

MODEL

The models were made to scale 1/60 and were fitted in a long masonry flume one below the other and discharges were measured in a well volumetrically. The model of the future design was fitted 27 feet from the head of the flume and that of the present design 28 feet downstream of this. The models were kept at such levels with respect to each other and the flume was provided with such stilling arrangements that the approaches to both models were smooth and straight. At the end of the flume was a calibration well 12 feet diameter, 6 feet deep in which the water passing over the models could be collected and measured. Supply at the head of the flume was controlled by sluice valves and kept constant by means of overflow arrangements. The depth of flow over each model was measured in a guage well located 5 feet upstream by means of pointer gauges having least count $\frac{1}{1,000}$ foot.

(10) Irrigation Research Station, Madras, Annual Report, 1947, pages 48—50.



Coordinates			
Present profile		Final profile	
X	Y	X	Y
0·0	16·68	0·0	16·68
0·0	16·68	0·0	6·8
6.5	0·39	6·5	0·39
8·3	0·0	8·3	0·0
14·0	1·2	14·0	1·2
17·0	2·79	17·0	2·79
17·5	13·3	20·0	4·89
Lie on the arc of a circle connecting points (17·5,13·3) & (43·67,40·0)		26·0	10·77
		32·0	18·66
		38·0	28·65
		54·5	60·5

Present profile		Final profile	
Net head in feet	C_d	Net head in feet	C_d
7·57	3·58	7·51	3·603
9·13	3·643	9·07	3·679
12·02	3·787	11·98	3·814
14·09	3·851	13·96	3·903
15·53	3·887	15·41	3·933

Figure 6C.11:- *Showing coefficient of discharge [Lower Bhavani Spillway Dam]*

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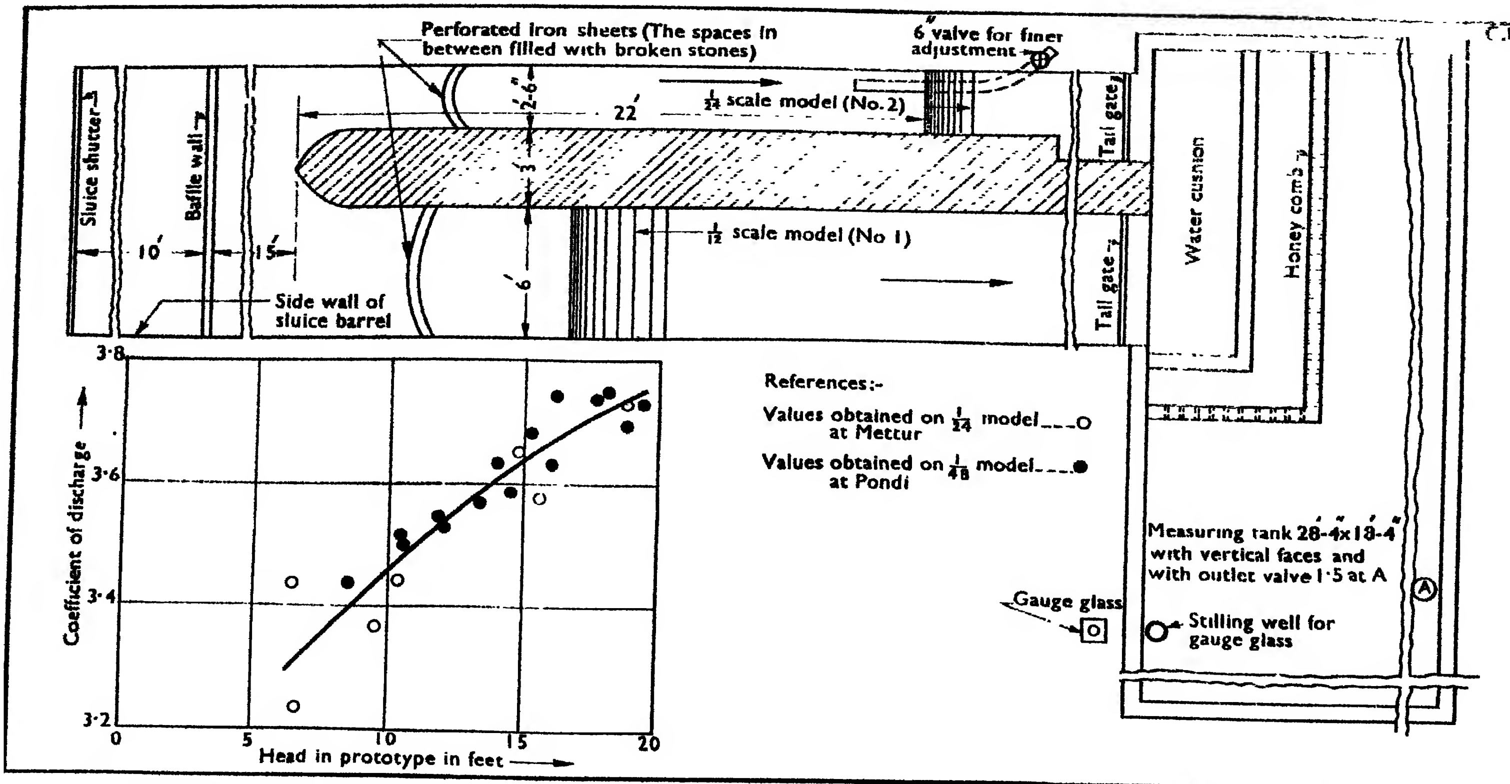
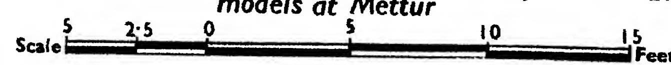


Figure 6C.12:- *Tungabhadra Spillway Dam – coefficient of discharge, Layout of $\frac{1}{24}$ and $\frac{1}{12}$ models at Mettur*



OBSERVATIONS

For any particular experiment a certain discharge was let into the flume and kept steady. The depths over the two models were measured and the time taken for filling a certain depth of the measuring well was noted on a stop watch. Observations corresponding to overflow depths of 5 to 15 feet were made and repeated many times. Allowance was made for head due to velocity of approach.

One set of values is furnished in the table and in the graph accompanying Figure 6 C. 11 Referring to these the following inferences were noted.

CONCLUSIONS

(*a*) For the final profile, which follows Creager's design, the co-efficient of discharges increased from 3·6 for a head of 7·5 feet to 3·93 for the full designed head of 15 feet.

(*b*) For the present profile the co-efficient rose from 3·56 to 3·88 for the same range. The reduction was due to the departure from the Creager's profile.

(*c*) For design purposes a value of 3·88 could be adopted.

(*d*) The results could be treated as basic and adopted for similar designs.

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(9) CO-EFFICIENT OF DISCHARGE FOR TUNGABHADRA DAM [11]

ABSTRACT

Studies made at Poondi on models of Tungabhadra Dam to scale 1/48 have been described in the Annual Report for 1945. Similar studies made at Poondi on small models of Ramapadasagar Dam and Lower Bhavani Dam can be seen discussed in the Annual Reports for 1946 and for 1947. In order to see how far these results are affected by scale effect two large models to scale 1/12 and 1/24 respectively were constructed in one of the high level sluice barrels of the Mettur Dam and tested.

MODEL

The models 1/24 and 1/12 were constructed in one of the high level vents of the Mettur Dam *vide* Figure 6 C. 12. A divide wall three feet wide was constructed inside the tunnel 10·5 feet wide dividing the tunnel into two flumes 5 feet and 2·5 feet wide for the 1/12 and 1/24 scale models respectively. Side walls were extended beyond the ventway downstream to provide the necessary lengths for the flumes. At the end of the flumes was a measuring tank 28 feet 4 inches × 18 feet 4 inches in plan, specially constructed for quantitatively measuring the water passing over the model.

The nose of the divide wall was 25 feet from the sluice gate and this intervening space served as a forebay to the model. There was a baffle wall 1·5 feet thick across the vent and 10 feet away from the shutter in the forebay for stilling the shooting stream. Further stilling was effected by suspending screens, built of two perforated steel with the space between filled with granite metal of two inches size. There were stilling arrangements in the measuring tank too, consisting of a water cushion and honeycomb walls.

([11]) Irrigation Research Station, Madras, Annual Report, 1947 pages 51—53

The model was built in masonry and plastered over smooth with cement plaster. The side walls of the flume were also plastered smooth for distances of five feet upstream and downstream. For finer adjustments of head over the crest of the model a six inches pipe outlet communicating with the upstream was built in the body of the model at the base and was controlled by a six inches valve at the outlet.

The head over the crest was measured with a pointer gauge reading to 1/1,000 foot. For noting the water level in the measuring tank a stilling well consisting of perforated screens and communicating the water level to a gauge glass outside, was used. The water in the measuring tank could be emptied by a 18 inches diameter valve.

EXPERIMENTS

Water was slowly admitted to the forebay till it just spilled over the crest. Then the supply was cut off and water upstream emptied by the six inches diameter pipe till the flow over the spillway stopped and water stood at the crest level under static condition. The water level elevation which was also the crest level was then recorded. Water was then admitted to the model corresponding to 20 feet overflow on top of dam in prototype. The required elevation was obtained by operating the sluice gate in combination with the six inches outlet. When steady conditions of flow were obtained the 1·5 feet diameter outlet valve of measuring tank was closed. As the level of water in the measuring tank rose up, the time required to collect one foot depth of water in the tank was noted. The experiment was repeated a number of times and the mean value taken for computations. Knowing the exact diamensions of the measuring tank and the time required to collect one foot depth of water in it the actual discharge over spillway was computed. The value of C_d was then calculated from the equation :

$$Q=C_d \left\{(h+ha)^{3/2}-ha^{3/2}\right\}$$

Here Q=actual measured discharge.

l=2·5 feet.

$$ha=\frac{v_1^2}{2g}$$

v_1=velocity of approach.

$$=\frac{q}{p+h}$$

The experiments were repeated for heads varying from 20 feet to 5 feet over the spillway.

DISCUSSION OF RESULTS

The co-efficients of discharge obtained on the model for various heads ranging from 5 feet to 20 feet are given in Table 6 C. 9. The values obtained on the $\frac{1}{48}$ model in the Poondi experiments of 1945 are also noted side by side for ready comparison. All these are plotted in Figure 6 C. 12 the points relating to the $\frac{1}{48}$ model being shown by circles. All these points overlap one another and a single curve is fitted to cover them. A value of 3·75 for the co-efficient for the maximum head was found suitable. No "scale effect" was indicated.

TABLE 6 C. 9

Tungabhadra Dam—Models and Mettur—Co-efficient of Discharge.

Serial Number	Head over dam in feet	Co-efficient of Discharge: Values obtained on $\frac{1}{48}$ model at Poondi	Co-efficient of Discharge: Values obtained on $\frac{1}{24}$ model at Mettur
1	6·57	..	3·436
	8·688	3·439	..
2	13·248	..	3·441
	10·70	3·50	..
3	15·79	..	3·575
	15·40	3·689	..
4	19·39	..	3·74
	19·36	3·715	..

The values obtained on the $\frac{1}{24}$ scale model at Mettur and on the $\frac{1}{48}$ scale model at Poondi have been furnished above. The $\frac{1}{12}$ model at Mettur turned out to be too unwieldy and no satisfactory observations could be made thereon. Similarly the $\frac{1}{96}$ model attempted at Poondi was seen to be affected by capillary and other effects viciating the readings. The results obtained on $\frac{1}{48}$ and $\frac{1}{24}$ scale models worked out independently at Poondi and Mettur showed good agreement and no scale effect. Results obtained from one model of moderate size (say $\frac{1}{36}$ to $\frac{1}{40}$) for these dams of height about 100 feet and overflow depth=about 20 feet could, therefore, be relied upon for design purposes.

(10) RAMAPADASAGAR SPILLWAY DAM—EFFECT OF PIERS ON CO-EFFICIENT OF DISCHARGE [12]

INTRODUCTION

Two designs of piers for the Ramapadasagar Spillway Dam were tested on models to find out the effect of having the pier cut waters projecting upstream of dam face and supported by corbelling on the co-efficient of spillway discharge. The designs are shown in Figure 6 C. 13. In one design the cutwater edge was flush with the upstream vertical face of the dam, while in the other the entire cutwater was projecting upstream of the dam face.

THE MODEL

Attempts were first made to evaluate the actual co-efficients of discharge for different heads on models having the two kinds of piers with a view to compare them and then find out the difference. A few experiments were made on a $\frac{1}{60}$ model (same as the one used for finding the co-efficient of discharge). Here the discharge was gauged by means of a precalibrated knife edged weir. It was found out that the accuracy of the values obtained was insufficient for the particular study.

Next a model to 1/100 was constructed within the Research Station gauging flume 3 feet $\times$ 3 feet $\times$ 60 feet and discharge was measured quantitatively in the gauging well. Even this was found to be insufficient to give the accuracy desired.

Finally, therefore, two models of the spillway without piers were constructed to scale 1/100 within the long measuring flume. These were made exactly similar and were kept 28·5 feet apart. The first model was constructed at a level 0·6 foot higher than the second so that when any desired level of water was maintained on the latter's crest, the former would not get submerged, nor have its modularity affected in any manner. Arrangements were also made to still the disturbance in the water discharging over the first model effectively before it reached the second. These are all shown in Figure 6 C. 13.

When all the above arrangements have been perfected a number of experiments were made allowing the same water to flow over both the models and taking the depth of flow over each model. It was seen that identical values were obtained on both. This was taken as 'proving the model'.

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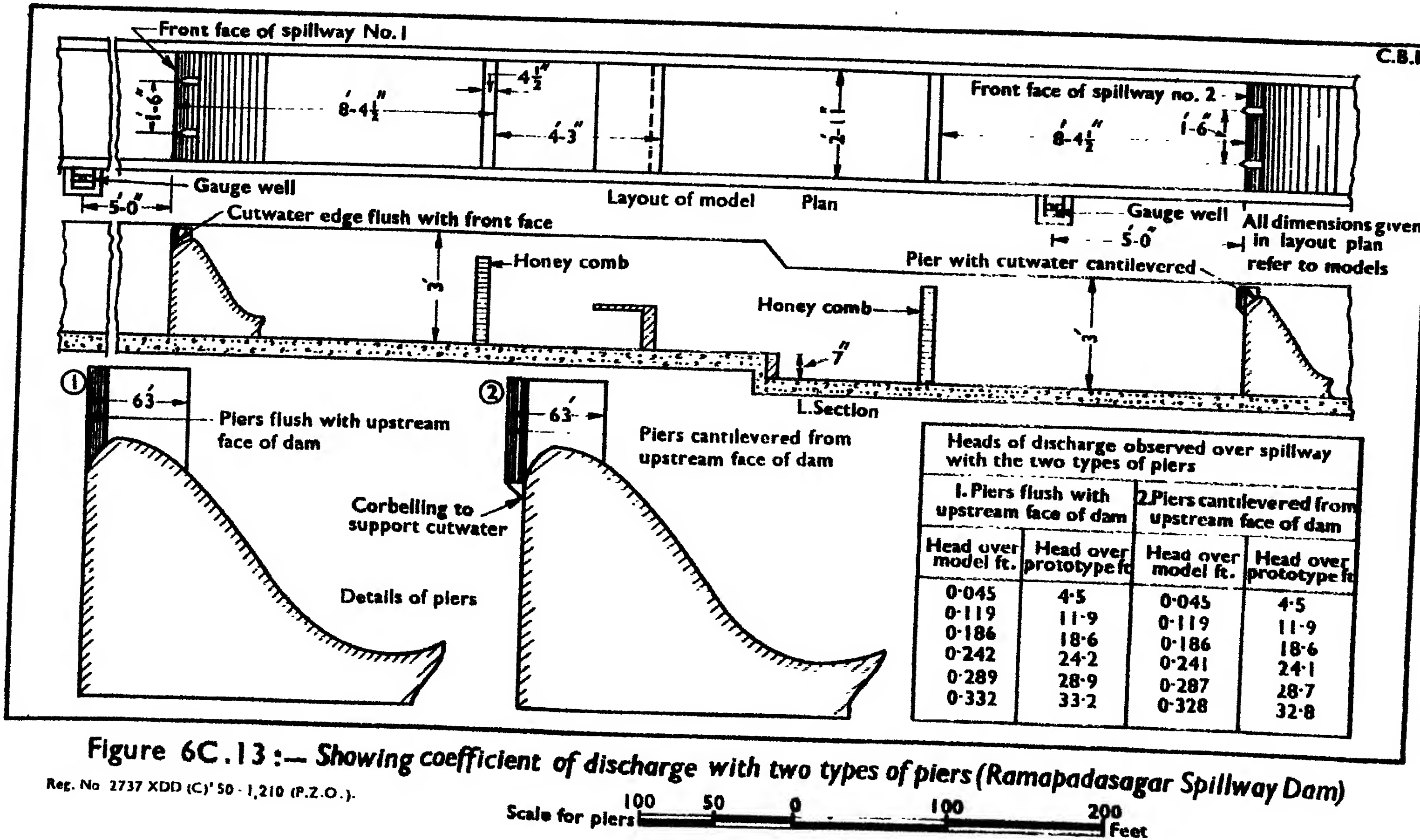
[12] Irrigation Research Station Madras, Annual Report, 1947, pages 53 to 55.



Heads of discharge observed over spillway with the two types of piers			
1. Piers flush with upstream face of dam		2. Piers cantilevered from upstream face of dam	
Head over model ft.	Head over prototype ft.	Head over model ft.	Head over prototype ft.
0·045	4·5	0·045	4·5
0·119	11·9	0·119	11·9
0·186	18·6	0·186	18·6
0·242	24·2	0·241	24·1
0·289	28·9	0·287	28·7
0·332	33·2	0·328	32·8

Figure 6C.13 :— *Showing coefficient of discharge with two types of piers (Ramapadasagar Spillway Dam)*

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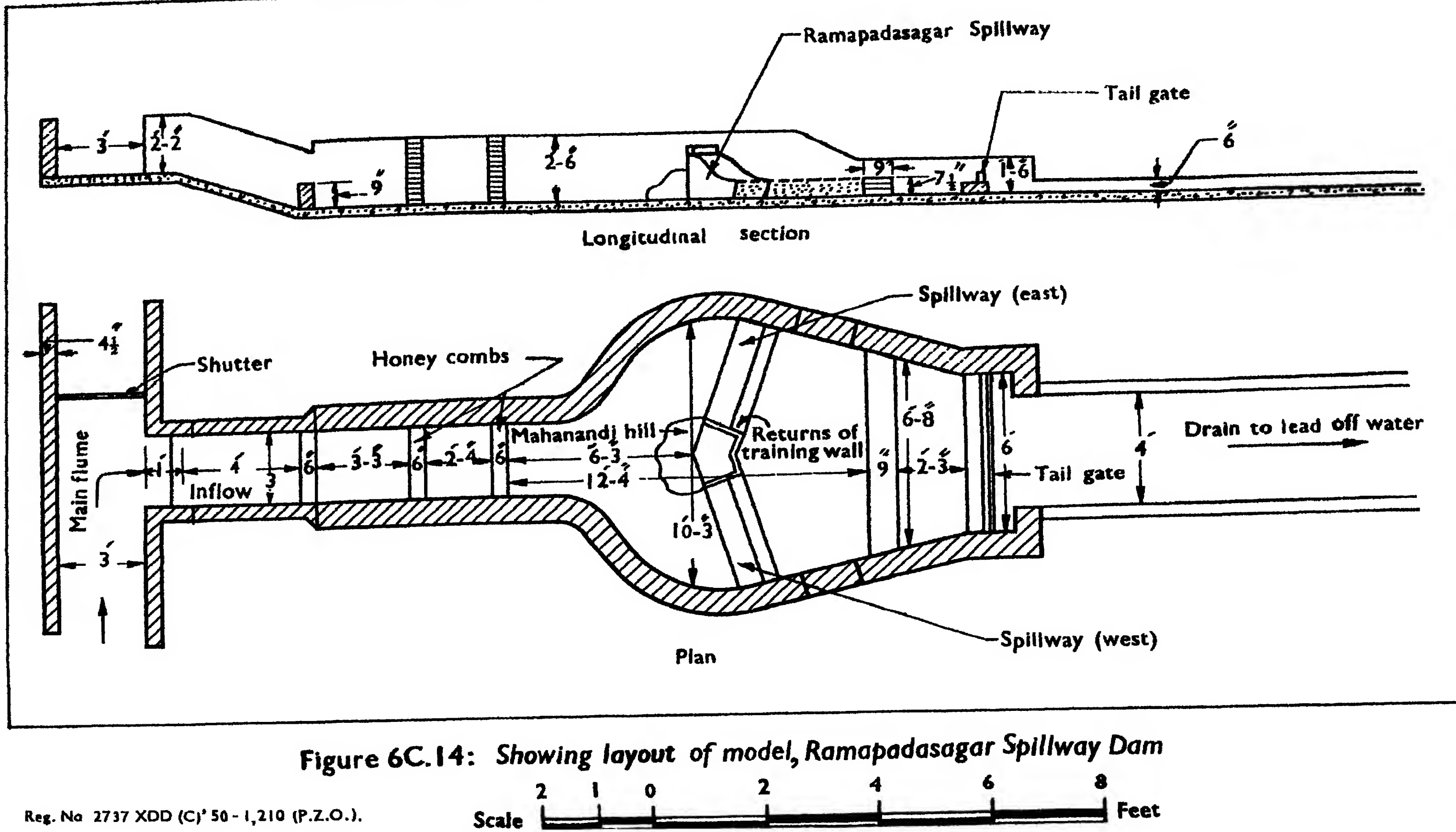

Figure 6C.14: *Showing layout of model, Ramapadasagar Spillway Dam*

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Next the final experiments were carried out to see the difference in discharge produced by the two types of piers. On the No. 1 model two piers were built with their cutwater edged flush with the front vertical face of the dam and on the No. 2 model two piers were built with their cutwaters projecting upstream. For the same water passing over both models, the depths of flow were measured. These are tabulated in Figure 6 C. 13.

These values show the difference clearly. For heads up to about 24 feet (prototype) there was no difference. For heads exceeding 24 feet there was a slight difference and this difference steadily rose with increase in head. For the maximum designed head of 30 feet the difference was about 0·3 foot. This worked out to one per cent. and was not appreciable.

CONCLUSION

By extending the pier cutwaters upstream the co-efficient of discharge could be slightly increased because of the improvement in the approach stream lines. The increase, however, was very small and could not be taken into consideration for design purposes.

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(11) SPILLWAY TRAINING WALLS FOR THE RAMAPADASAGAR DAM[13]

INTRODUCTION

The spillway portion of the Ramapadasagar Dam was designed in two sections forming a hockey stick alignment about the Mahanadi Hill. On this hill and at the flanks were to be non-overflow dams. Reinforced concrete training walls were proposed at the ends of spillways both at the Mahanadi Hill and at the flanks. The alignment of the training walls and the pressures exerted on them by the overflowing water were taken up for test on hydraulic models.

THE MODEL

The model was constructed in a masonry tray 40 feet × 12 feet × 3 feet inside and having the necessary arrangements for water-supply at constant head and for measuring the same. The layout of the model is shown in Figure 6 C. 14. The model was constructed to scale 1/150 natural and comprised of the Mahanadi Hill and of four spans of the spillway on either side of the hill. The cistern walls on the upstream side were so aligned as to ensure a normal

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[13] Irrigation Research Station, Madras, Annual Report, 1947, pages 55- 58.

approach towards both the spillways which will be the case in the prototype. The river bed downstream for a length of 9 feet≡1,350 feet was included and was moulded level with well packed sieved sand of 24—30 mesh per inch grade to a depth of 0·75 foot. The top of sand was kept at level corresponding to plus 20 in the prototype. A tail bay and a tail gate were provided at the end for regulating downstream water levels.

The training walls were made in well-polished teakwood and fitted up as shown in Figure 6 C. 14. In the first design the returns were parallel to and coincident with the end of the buckets. In the second design a stream lined, *i.e.*, a semi-elliptical shape was given to the returns The training walls proper (on both sides) were at right angles to the axes of the spillways and had vertical faces on the water sides. On each of these walls a set of 12 piezometer points were introduced and were tapped off by tubings and connected to a manometer set up outside.

OPERATION OF MODEL

Apart from a large number of qualitative tests, two experiments were made on the model; (*i*) with the original design of returns and (*ii*) with the modified design having the streamlined returns. In both experiments M.W.L. was maintained upstream and downstream. The run in each case was for a period of one hour. The scours formed on the bed were traced and pressures on the manometer set up were read off and recorded.

OBSERVATIONS

With the original design of the returns there were eddies formed near the returns. When the returns were streamlined, these got eliminated. Excepting for this, which by the way was not quite serious, there were no bad eddying for any flow, and the flow was satisfactory. The streamlining given did not involve any considerable extra concrete in superstructure or foundations. Any further lengthening of the wall downstream would involve costly foundation, which was found unnecessary.

CONCLUSION

The original designs of the training walls and returns were found generally satisfactory. The right angular bends for returns caused a certain amount of eddying which was, however, not harmful. Even this could be eliminated by having a semi-elliptical alignment for the returns.

As regards pressures exerted on the training walls, further studies on a large scale model were necessary *before final conclusions* could be drawn.

(12) STILLING BASIN FOR LOWER BHAVANI SPILLWAY DAM [14]

ABSTRACT

Experiments for obtaining satisfactory designs of stilling basins for moderately high dams were continued during 1947 with special reference to the Lower Bhavani Spillway Dam. In this connection two series of experiments were carried out : the first series on a model of the 15 feet overflow section and the second series on a model of the 10 feet overflow section. These are described in this article.

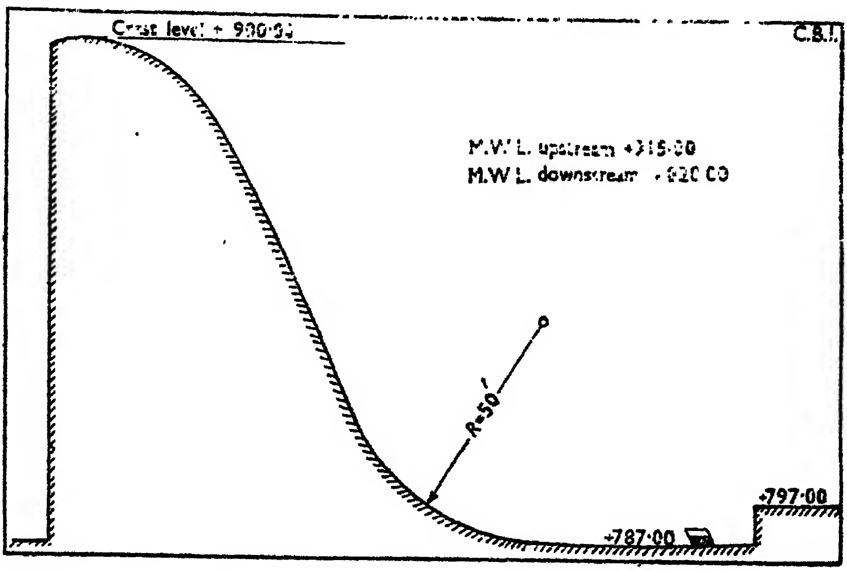


Figure 6 C.15 : *Showing 15 feet overflow section of Lower Bhavani Spillway dam.*

HYDRAULIC PARTICULARS

The design of the spillway dam adopted for the present experiments is given in Figure 6 C.15. The hydraulic particulars relevant to the study are furnished below :—

River bed (average)	+797·0.
Crest of dam	+900·0.
M. W. L. upstream	+915·0.
M. W. L. downstream	+820·0
Discharge per foot run	225 cusecs.

(14) Irrigation Research Station, Madras, Annual Report, 1947, pages 65—80.

MODEL

The model was made in polished teakwood to scale 1/120 and fitted up in the Laboratory Hydraulic Flume 40 feet×2 feet×2 feet. Downstream of the model sieved sand of 24—30 mesh per inch grade was packed to a depth of ¾ foot. All experiments reported here were made for the condition corresponding to M. W. L. upstream and M. W. L. downstream. Qualitative observations were, however, made for other conditions also with a view specially to see the range of effectiveness of the design in creating and maintaining a proper standing wave for various possible conditions of flow. Water-supply to the model was drawn from the overhead tank, measured over a precalibrated weir in the flume and properly stilled. The downstream water levels were adjusted by a tail gate. The runs were made for two hours in each case, which period was found sufficient to give a stabilized bed downstream. All measurements of water surface profiles and bed scours were made by point gauges reading to a thousandth of a foot.

SCOPE OF STUDIES

The present studies in some respects were a continuation of previous ones made on models of the Tungabhadra, the Ramapadasagar, and the Malampuzha Spillway Dams; but there were considerations that had a special bearing to the Lower Bhawani Dam. Unlike the Tungabhadra and the Malampuzha Dam there is no hard rocky bed at the Lower Bhavani Dam site and strong apron in continuation of stilling basin would be a costly work. Nor was it possible to allow for a deep scour as in the Ramapadasagar Dam where the river bed is sandy and the dam foundations go deep down to rock. What was aimed at was to obtain a stilling basin within which dissipation of energy would be secured to the maximum possible extent, so that any apron that might be had downstream of the basin should not have to withstand high erosive action.

This was sought to be obtained by ensuring the formation of an "efficient standing wave" within the stilling basin for all possible flow conditions. A hump or a 'boil' in the standing wave rising above the tail water causes a dive and a secondary wave. In the absence of a strong apron, this is bound to create a bad scour, the scour being almost a reflection of the boil, see Figures 6 C.16(A), (B) and (C) in which the standing waves have such boils and deep scours are formed consequently. On the other hand in Figures 6 C.16 (K), (L) and (N) no humps are seen and no bed scours. In the case of simple stilling basins it was seen that a single sill of any type could be designed to give a "perfect standing wave" only within a limited range of flow conditions. A slight deviation in the quantity of discharge or in the tail water gets a hump in the standing wave or gets the nappe washed clear out of the basin. Provision of one or more lines of splitter blocks inside the basin (called "floor blocks" to differentiate from similar blocks used in other positions) was found to improve the standing wave and to retain this condition for a very wide range of flow conditions. The major part of the present studies was, therefore, devoted to the selection of a suitable type of floor blocks.

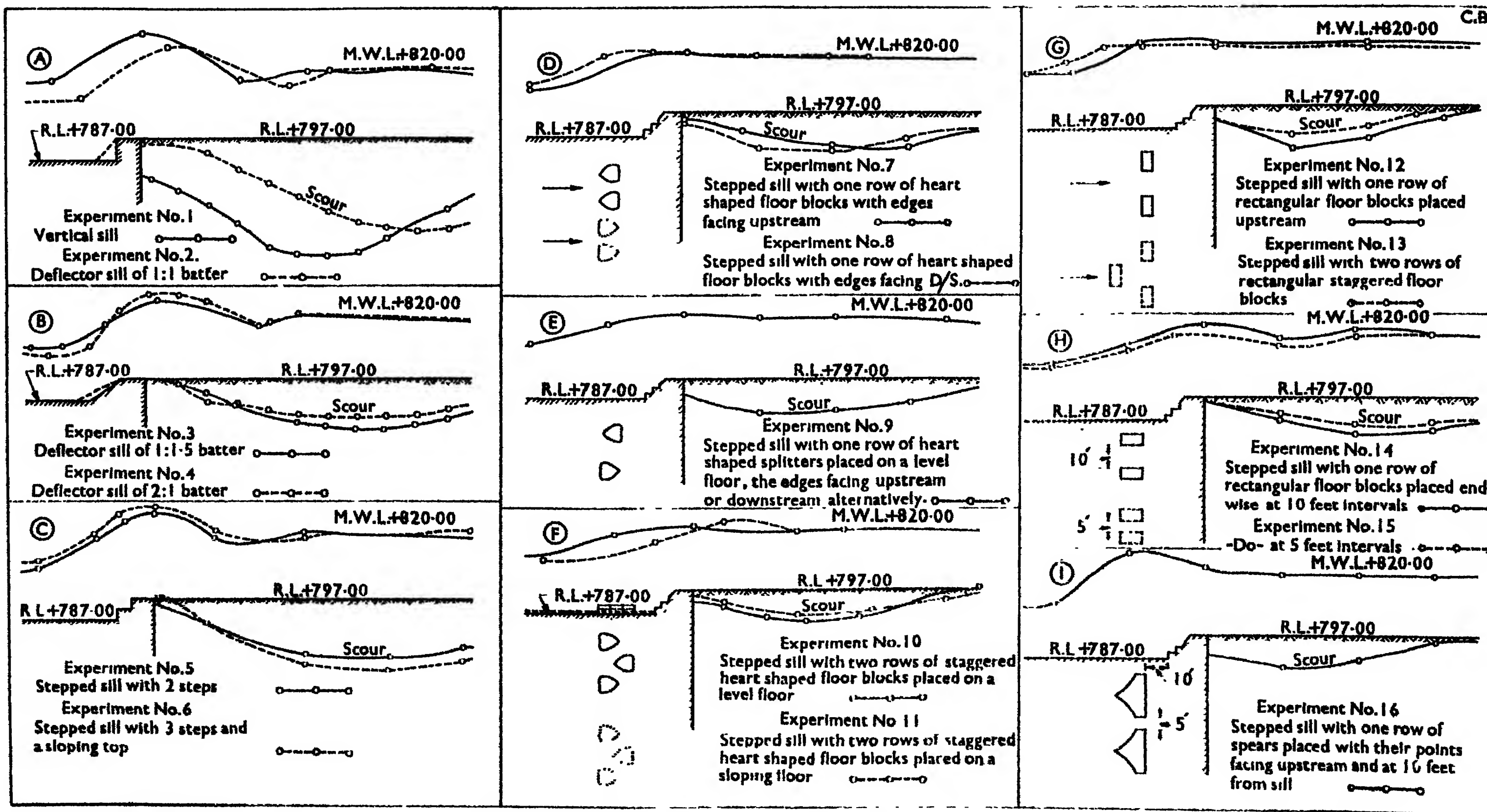
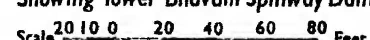


Figure 6C.16 :- *Showing lower Bhavani Spillway Dam*



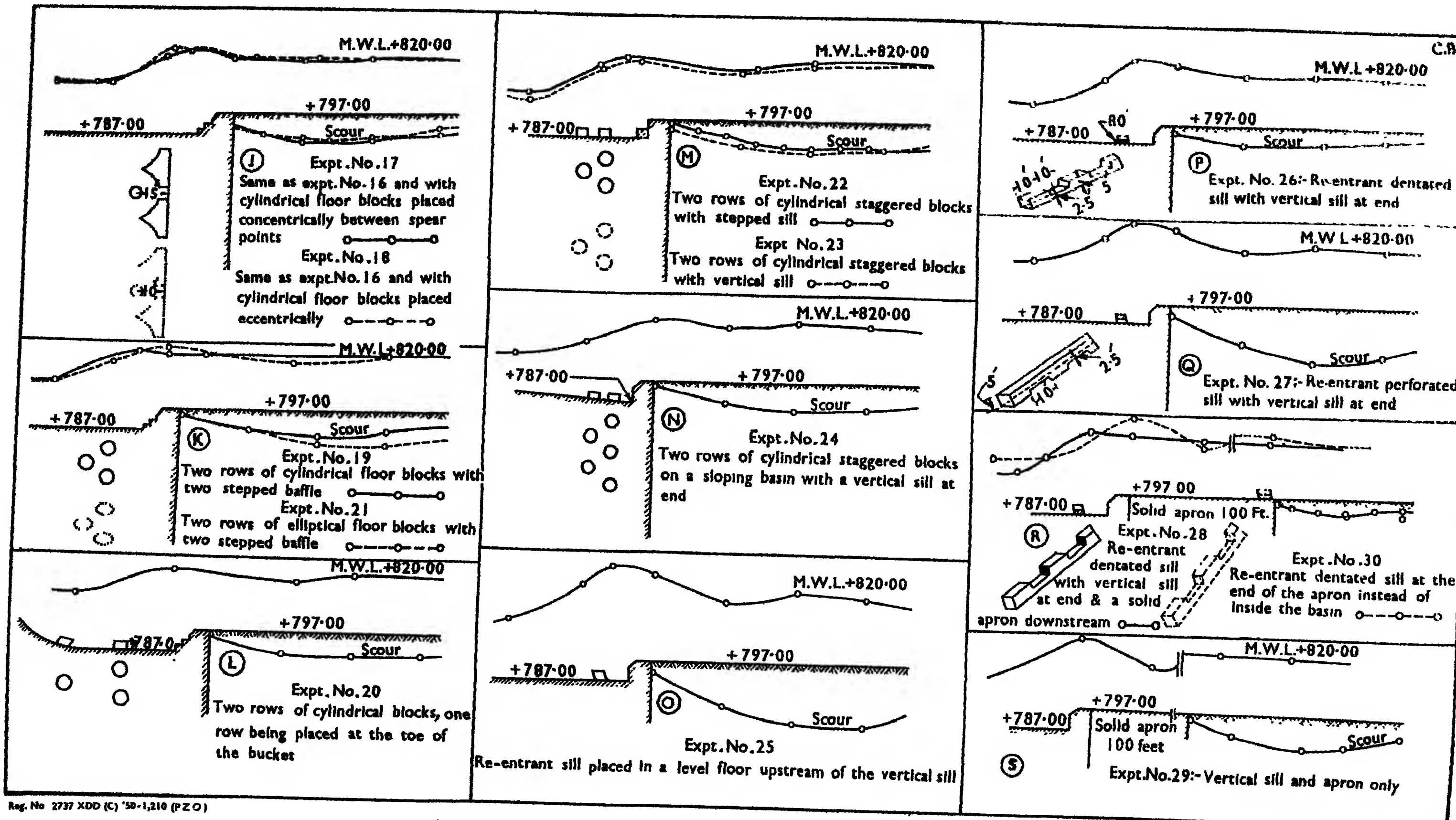


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Figure 6C.16 (Contd) :- *Showing lower Bhavani spillway dam*

DISCUSSIONS OF RESULTS

Experiments numbers 1 to 6 were made with a stilling basin 50 feet long and a sill at the end. Different types of sills were tried. The sloping sills were better for the normal conditions of flow but with low tail water levels, the standing wave got washed out of the cistern. With the vertical and stepped sills this danger could be eliminated, but the standing waves always showed a boil and the scours were deep.

In Experiment No. 29 an apron 100 feet long was added in continuation of the stilling basin. This slightly reduced the scour. In Experiment No. 30 an end sill was introduced at the end of the apron which reduced the scour still further. The design tested in Experiment No. 30 is fairly satisfactory provided the apron is made strong enough to withstand the wear caused by the flow issuing beyond the sill, in which part of the energy would remain undissipated.

In Experiments Nos. 25, 26 and 27 a continuous re-entrant sill, a continuous but dentated re-entrant sill and a continuous but perforated re-entrant sill were successively tried. The first and third were unsatisfactory because they practically acted as the main sill and did not function as " Splitters ". In Experiment No. 25 the portion between the re-entrant sill and the main sill got filled up by the sand washed into the cistern within a minute of starting the experiment. The design tested in Experiment No. 26 behaved well.

In all other experiments some types of floor blocks were introduced and tested. These generally were satisfactory. Some were better than others. The floor blocks functioned as dissipators for several reasons, *viz.*, (*a*) direct impact on the blocks, (*b*) frictional effect of the sides, (*c*) splitting up of the main jet into smaller jets, (*d*) interaction of these jets amongst themselves. The overall effect was that practically all the surplus energy was dissipated within the hydraulic jump itself and there was no need left for having a substantial downstream.

The various shapes of blocks tested were designed to emphasize one or more of the effects mentioned above. Thus rectangular blocks proved the best design for direct impact, the spear shaped blocks were good splitters, the elliptical blocks encouraged counteraction of jets and almost all designsa cted as friction blocks. The cylindrical blocks placed staggered in two rows (Experiment No. 23) proved to be a good combination and seemed best. Its adoption may, however, be objected to on the ground that it will be difficult to get good anchoring of these blocks in many situations. Where it is so the re-entrant dentated sill can be adopted which is equally good from hydraulic considerations. The weak point about this is the difficulty in maintaining the sharp edges intact against the violent impacts they have to withstand.

CONCLUSION

A stilling basin 50 feet long 10 feet deep having a 40 feet radius bucket upstream and a vertical sill downstream followed by a 100 feet apron is recommended. In order to increase the efficiency of the design a system of floor

blocks consisting of two rows of cylindrical staggered blocks or merely of one continuous dentated re-entrant sill of the dimensions indicated may be provided inside the stilling basin. The standing wave with such a design will not rise more then three feet above the M. W. L. downstream and the scours will not be deeper than eight feet below the apron level.

Individual floor blocks are always difficult to anchor so as to withstand the enormous pressure caused by the shooting flow impinging on the blocks. The Chief Engineer for Irrigation, therefore, suggested a system of Tee shaped blocks having all the benefits of simple blocks and also the stability and strength afforded by the buttress walls forming the legs of the Tees. This device was tested and developed in a second series of experiments on a model of the Lower Bhavani Spillway Dam. For these experiments a model of the 20 feet overflow section was adopted.

HYDRAULIC PARTICULARS

The design of the spillway is furnished in Figure 6 C.17 (A, B, C, D). The hydraulic particulars are below :—

River bed average	+ 797·0
Crest of spillway	+ 905·0
M. W. L. upstream	+ 925·0
M. W. L. downstream	+ 830·0
Discharge per foot run	662·5 c/s.

MODEL

The model was made to scale $\frac{1}{30}$ and was fitted up inside the 40 feet × 2 feet × 2 feet hydraulic flume. In all details the set up and operation were identical to the one adopted in the first series of experiments.

The design found to be the best is shown in Figure 6 C.17. (A). This was obtained early in the course of the experiments. It consisted of a stilling basin 30 feet long beyond toe of bucket × 10 feet deep, having a simple vertical sill at end and a single row of Tee shaped blocks in the basin. The rectangular blocks were parallel to the sill, size 8 feet × 4 feet × 6 feet, spaced 16 feet clear apart and buttressed on to the end sill by longitudinal blocks 2 feet × 5 feet and 20 feet long. The front blocks were observed to split the shooting jet into two parts. One part directly striking on the blocks got thrown up and fell down in the form of a canopy ; the other part flowing between two adjacent blocks expanded laterally between the buttresses and was confronted by the end wall of the basin. The combined result of all these actions and counteractions was that a very satisfactory amount of dissipation and deflection was produced inside the basin. The design was decidedly an improvement in many respects over the one obtained in the first series of experiments.

In a number of experiments various shapes of blocks all butting against the sill were tried. All these were unsatisfactory as they failed to allow for the lateral expansion of the jet passing through the gaps between blocks. This resulted in residual energy creating a boil and/or in the washing out of the standing wave for low tail water stages. Other experiments were mostly intended to obtain optimum dimensions for the various parts of the designs.

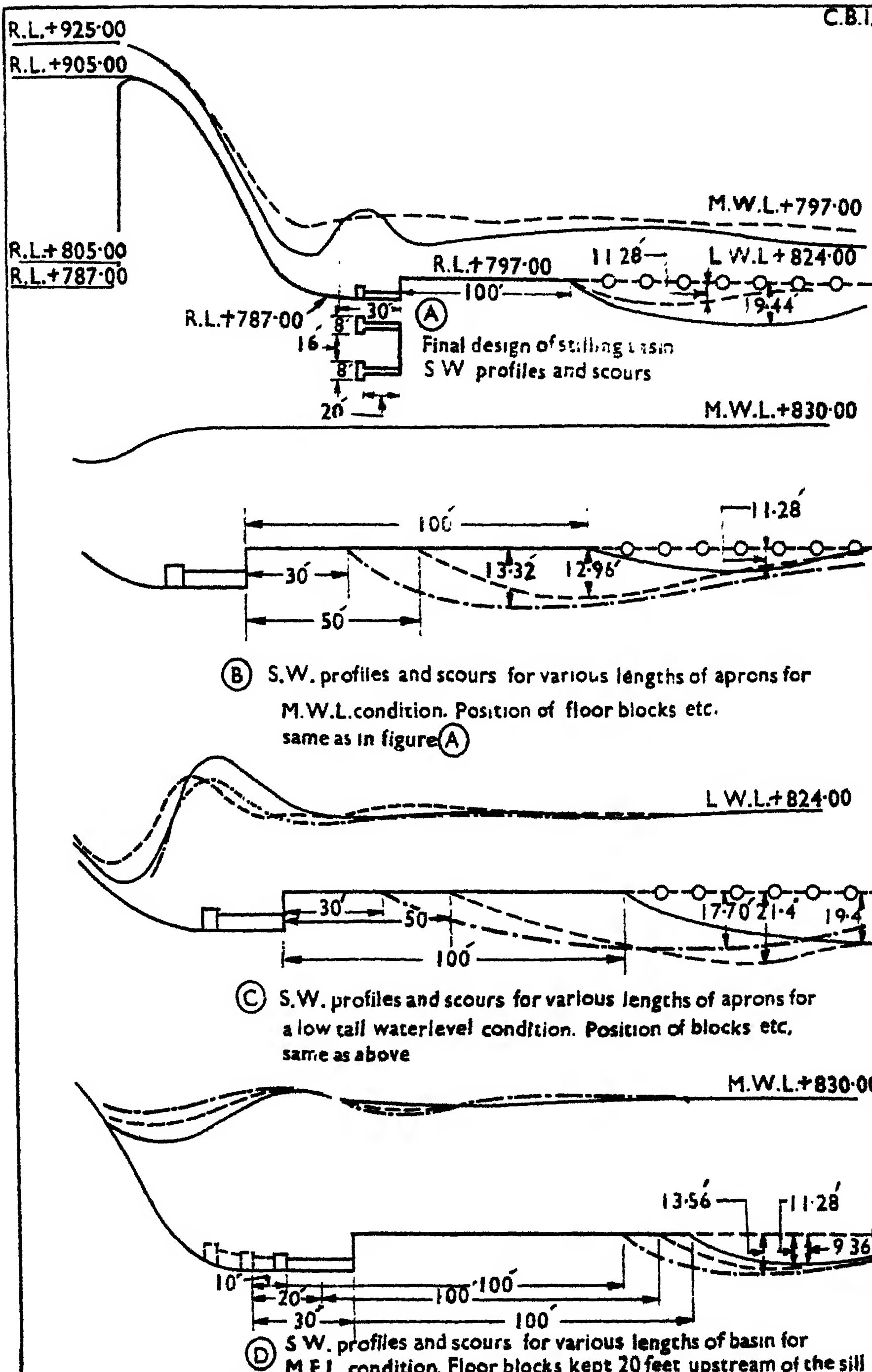

Figure 6C.17 :- *Lower Bhavani Spillway Dam 20 feet overflow section*

A — Scale 40 20 0 40 80 120 160 Feet

B-D — Scale 20 10 0 20 40 60 80 Feet

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CONCLUSION

A stilling basin consisting of a simple bucket and sill and a row of Tee shaped blocks of the dimensions given in Figure 6 C.17(A) was found the most efficient. It yields an ideal standing wave for a wide range of fluctuations in the water levels. The crest of the standing wave was only 0·24 above the tail water level for M. W. L. conditions. The maximum scour was only 11·28 feet. The design was cheaper than anything so far known and having about the same efficiency. Further studies for finding out the pressures on the various parts of the blocks and sill and to locate cavitation, if any, are in progress.

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(13) TUNGABHADRA SPILLWAY DAM—THREE DIMENSIONAL MODLES [15]

Further tests were made during 1947 on various models for obtaining refinements of design with reference to fuller data that became available during the year. As no special features were involved in these tests they are not described here.

Two sets of design evolved from the studies on sectional models were finally testedson full three dimensional models. This requires special attention and is discussed below.

DESINGS

Design No. 1 consisted of a spillway of over all length=1,810 feet made up of 26 spans of 60 feet each and 25 piers of 10 feet each. The overflow depth was 20 feet. Design No. 2 had an overall length of 3,505 feet having 50 spans. Length of span and width of piers were the same as in Design No. 1 except for three abutment piers which were 15 feet each. The overflow depth in this was only 15·75 feet. As the spillway extended to the high margins on the right of the river, training walls were provided downstream to divert the discharge towards the river proper. The downstream aprons in both designs war at +1,524 in the river portion. In Design No. II the aprons on the margin were higher by 16 feet and 26 feet in the two sections.

The hydraulic particulars are furnished below :—

Description	*Design I*	*Design II*
Flood discharge	650,000 cusecs	..
Full reservoir level	+1,630	+1,630
Crest of spillway	+1,610	+1,614·25
Number of spans	26	50
Discharge per foot length	430 cusecs	218 cusecs
Downstream floor level	+1,524·0	+1,524·0 +1,540·0 +1,550·0
Downstream M. F. L.	+1,549·0	

([15]) Irrigation Research Station, Madras, Annual Report, 1947, pages 80—84.

MODEL

The two designs were constructed to scale 1/120 on two sides of a cistern 40 feet × 40 feet × 2 feet. Water was let into the cistern at one corner and was controlled and regulated by overflow and gate arrangements and the discharge was measured on a standard standing wave flume. Downstream of each model dam was formed the river proper with the bed, margins, islands, *etc.*, according to cross-sections actually taken at site for the purpose. The mobile portion of the river bed was formed with sieved sand of 24—30 mesh per inch size with the help of pegs fixed on the bed and having their tops adjusted to correct levels. For each model there was a tail bay and a tail gate for controlling the downstream water levels and arrangement for feeding the model from the tail end before starting an experiment.

EXPERIMENTS

Experiments were carried out for M. W. L. conditions upstream and downstream. The models were initially filled up to downstream M. F. L. from the tail ends gradually so as not to disturb the beds and then the main cistern was filled and continuously supplied to maintain M. F. L. discharge. Under this condition the tail gates were adjusted to maintain the downstream M. F. L. and the run was continued for one full hour. During the run the flow characteristics were observed and at the end of the run the models were carefully emptied and the scours observed and measured.

OBSERVATIONS

During the course of operation of the models the following observations were made :—

(1) In Design No. I a standing wave was formed over the stilling basin for all conditions of flow. Under M. F. L. conditions the standing wave appeared uniform and steady and smooth flow ensued beyond the apron.

(2) In Design No. II there was shooting flow at the Rehbock's dentated sill for various flow conditions. Under M. F. L. discharge shooting flow persisted in the two sections on the extreme right covering 23 spans. Here the depth of downstream water level was insufficient to create a standing wave.

(3) At the end of the run scours were plotted. The general disposition of the bed was clearly more satisfactory in Design No. I.

(4) Design No. I was evidently more satisfactory. This could be taken as an indication that as far as possible, spillway length should be restricted to the river portion proper. By extending over marginal lands we loose the tail water depth which is so essential a factor in the formatnion of standing waves.

(14) HIGH COEFFICIENT WEIRS FOR IRRIGATION TANKS [16]

ABSTRACT

In order to discharge the maximum flood discharge of 13,300 cusecs over the existing length of 725 feet with a head of 2½ feet the Arjunpatla weir should have a coefficient of discharge of 4·64. Experiments were conducted to evolve a type of weir that will give a coefficient of discharge of 4·64 under a head of 2½ feet.

The maximum flood discharge of the [illegible] catchment area as calculated according to Ryve's Formula with C=800 is 13 300 cusecs. Assuming a coefficient of 3·10 for the existing weir it can discharge 8 830 cusecs under the proposed head of 2½ feet. In restoring the tank arrangement has to be made to discharge the balance of 4,470 cusecs. This can be done either by increasing the length of the existing weir by 375 feet or by altering the existing weir to give a higher coefficient of discharge. In this particular case it is found that it may be more economical and suitable to alter the [illegible] of the existing weir so that it could give a higher coefficient of discharge.

The first series of model experiments was conducted in Flume No. 1 which is one foot wide and 25 feet long. The first model was of ¼ scale and was made in concrete with pressure pipes inserted at various points on the circular crest and downstream glacis. This gave a coefficient of [illegible] for a head of 0·570 foot equivalent to 2·28 feet in the prototype.

Experiments were conducted in Flume No. 1 with various profiles having different crest diameters and glacis slopes.

At first the experiments were conducted with an upstream ground level one foot below the crest of weir as found at site. As these did not give the required coefficient of discharge the experiment was repeated for a depth of 1·5 feet which gave a coefficient of discharge of 4·50. This weir would be capable of discharging 12,900 cusecs and the balance of 400 cusecs could easily be taken care of by the flood absorption capacity of the reservoir.

The experiments in connection with the weir of Arjunpatla large tank indicated that the coefficient falls in value for heads above 2½ feet for the particular profile. Hence the profile of the Gagilla pur weir had to be altered :—

(*i*) To give a coefficient of 4·50 or every near it.

(*ii*) So that the value of the coefficient should not fall until a head of 3 feet is reached.

These experiments were conducted (in Flume No. 2, 3½ feet wide) on ¼ scale models one foot in length. At first a few parabolic profiles were tried and it was found that these gave coefficients of the order of 3·8 to 3·9. Hence the effect of increasing the crest radius in the circular profile was tried. Various crest radii of 5 inches, 6 inches and 7 inches were tried and it was found that a crest radius of 7 inches gave the required coefficient of 4·50 at a prototype head of 3 feet. These experiments were conducted with an upstream obstruction removal of 1½ feet below crest. Next the effect of reducing the obstruction removal to one foot was tried and the value of the coefficient fell to 4·25 at 3 feet head.

([16]) Hyderabad Engineering Research Laboratories, Annual Report, 1947, Pages 4—7.

(15) SIPHON SPILLWAYS FORMULAE [17]

In the design of siphon spillways, it is usually desirable that the jet should strike the river bed as far from the work as possible, in order to avoid bed scour close to the toe.

If the jet is taken to behave like a particle, the elements in the problem reduce to the following, Figure 6 C.18.

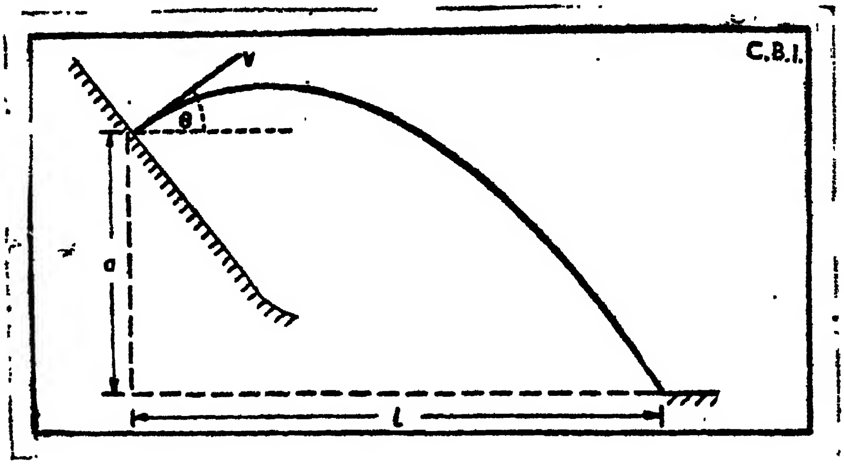

Figure 6 C.18 : *Reference diagram showing elements in the problem*

l =horizontal travel of the jet,

v =initial jet velocity,

θ =Initial inclination of the jet to the horizontal,

a =height of the nozzle above the river bed.

Naylor (1935, 'Siphon Spillways', pp. 41—42) has dealt with the problem.

He has stated (without proof), that, for l to be a maximumm

$$\text{cosec } \theta = \sqrt{\frac{2ga}{v^2+2}} \quad \ldots \quad \ldots \quad \ldots \quad (6C.1)$$

and then.......... $l = \frac{v^2}{g}$ Cosec $\left(\text{Sin } \theta + \sqrt{(\text{Sin } \theta + \frac{2ga)}{v^2}}\right.$ .. (6C.2)

(17) East Punjab Irrigation Research Institute, Amritsar, Annual Report 1947 pages 33—38.

Equation (6C.1) involves a radical, while (6C.2) is rather combersome and, in addition, requires t to be worked out first.

A simple expression for l directly in terms of v and a, and another for θ involving no radicals, were derived in the Institute during the period under report. Graphical solutions were also obtained, in order to obviate any reference to a table of trigonometric functions.

The initial jet velocity, Figure 6C.18, being v, at an angle θ to the horizontal, its horizontal component is $v \cos \theta$. If it takes a time t to reach the river bed, then since there is no horizontal acceleration.

$$l = vt \cos \theta \quad .. \quad .. \quad .. \quad .. \quad (6C.3)$$

The vertical component of the velocity is $v \sin \theta$ *upwards* and as the acceleration is g, downwards, and the vertical distance covered is a, in a downward direction, we get

$$-a = vT \sin t\theta - \tfrac{1}{2} gt^2 \quad .. \quad .. \quad .. \quad .. \quad (6C.4)$$

Eliminating T from (6C.3) and (6C.4)

$$-a = l \tan \theta - \frac{gl^2.}{zV^2} \operatorname{Sec}^2 \theta,$$

which may be re-written as

$$gl^2.\tan^2\theta - 2v^2l. \quad \tan \theta \, gl^2 - 2v^2a = 0 \quad .. \quad .. \quad .. \quad (6C.5)$$

As (6C.5) is a quadratic in $\tan \theta$, and θ is real, its roots must be real for which the condition is :

$$4V^4l^2 > 4gl^2(gl^2 - zV^2a),$$

or $$> g^2l^2 - 2V^2ga$$

or $$g^2l^2 < V^4 + 2V^2ga$$

or $$l < \sqrt{V^2 + 2ga}/g$$

Hence the maximum value of l is given by

$$l = V\sqrt{V^2 + 2ga}/g \quad .. \quad .. \quad .. \quad .. \quad (6C.6)$$

When this happens, the roots of (6C.5) are equal and so

$$2\tan\theta = 2v^2l/gl^2 \quad \ldots \quad (6C.7)$$

and

$$\tan^2\theta = (gl^2-2V^2a)/gl^2 \quad \ldots \quad (6C.8)$$

$$\tan^2\theta = 2\tan\theta/(1-\tan\theta)$$

from (6C.7) and (6C.8) $=2V^2l/2V^2a=l/a$ .. .. .. (6C.9)

Also

$$\text{Sec}^2 2\theta = 1+\tan^2 2\theta$$

from (6C.9) $=1+l^2/a^2$

from (6C.6) $=1+V^2(V^2+2ga)/g^2a^2$

$$=(V^2+ga)^2/g^2a^2$$

$$\text{Sec}\ 2\theta = (V^2+ga)/ga \quad \ldots \quad (6C.10)$$

$$\theta = \tfrac{1}{2}\tan^{-1}l/a = \tfrac{1}{2}\,\text{Sec}^{-1}\left(\frac{V^2}{ga}+1\right) \quad \ldots \quad (6C.11)$$

From Equation (6C.9) it is clear that, as l/a cannot exceed infinity, θ cannot exceed 45°.

The following numerical examples are given by way of illustration :—

(1) *Laggan Dam Spillway Siphon* :

This example is given at pp 41—42 of Naylor's 'Siphon Spillways'.

Here $V=50$, $a=64$.

Equation (6C.6) gives

$$l=50\sqrt{6596}/32$$

$=126\cdot9$, say 127 feet .. .. (6C.12)

Hence, from (6C.11)

$$\theta = \tfrac{1}{2}\tan^{-1}\frac{126\cdot9}{64}$$

$=31\cdot62°$, say $31\cdot6°$ .. .. (6C.13)

Naylor's values are :—

$\theta - 31{\cdot}6°$, and then l=128 feet.

(2) *Marikanve Dam (Mysore) Spillway Siphon* :

The data for this are given at pp. 342—343 of 'Irrigation Engineering' by K. R. Sharma.

Here $v=50$, $a=32$

Equation (6C·6) gives

$l = 50\sqrt{4548}/32$

105·4, say 105 feet .. .. (6C.14)

$\theta = \frac{1}{2} \tan^{-1} \dfrac{105{\cdot}4}{32} = 36{\cdot}55°$ say $36{\cdot}5°$

Sharma got $\theta = 37°$ and then l=132 feet.

The latter seems to be incorrect due to omission of the factor $\cos\theta=0{\cdot}8$ nearly.

(3) *Rasul Hydel Spillway (First design)* :

The details of this design are given in Figure 89, facing page 56 of the Annual

Report of the Punjab Irrigation Research Institute for 1945.

Crest R. L. =788·0

Nozzle R. L. =733·5 (at the centre line)

Downstream R. L.=706·1

Discharge coefficient=0·75

$v = 0{\cdot}75\sqrt{2\times32\times(788{\cdot}0-733{\cdot}5)}$

$= 44{\cdot}3$ feet per second

$a = 733{\cdot}5 - 706{\cdot}4 - 27{\cdot}1$

$l = 44{\cdot}3\sqrt{1962-1754}/32$

$$=84 \cdot 4 \text{ feet} \quad \ldots \quad \ldots \quad \ldots \quad \ldots \text{(6C.16)}$$

$$\theta = \tfrac{1}{2} \tan^{-1} \frac{84 \cdot 4}{27 \cdot 4}$$

$$=36 \cdot 0^{\circ} \quad \ldots \quad \ldots \quad \ldots \quad \ldots \text{(6C.17)}$$

The values in (6C.16) and (6C.17) agree with the designed 85·3 feet and 35·7°, within the limits of practical accuracy.

To obviate the necessity of calculating square roots or consulting tables of trigonometric functions, simple graphical solutions were also designed. These were based on exact theory and would give results accurate to within the limits of working error.

(a) *Solutions, for l and θ, Values of v and a known.*

The steps were Figure 6C.19 :

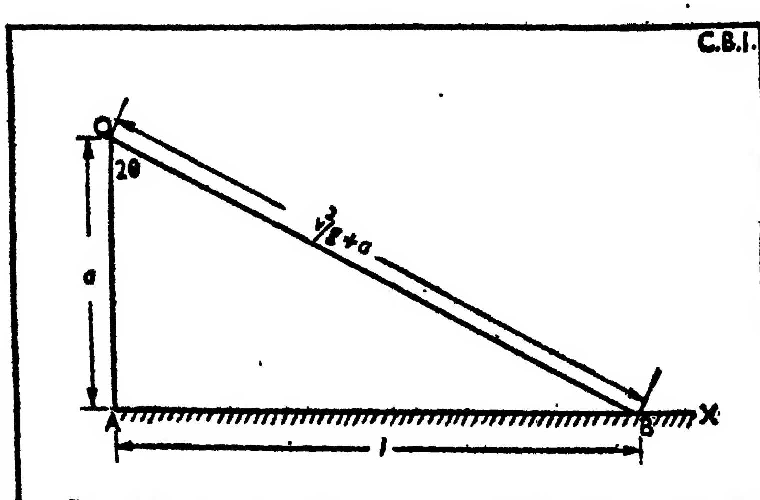

Figure 6 C.19 : *Showing geometrical construction for l and t.*

(i) Let $\dfrac{O}{a}$ be the nozzle and A the point below it on the river bed. Let $Oa=A$

(ii) With O as centre and radius equal to $\dfrac{V^2}{g}-a$, draw an arc to cut AX in B.

(iii) Join OB. Measure angle AOB.

(iv) Then AB equals l and angle AOB equals 20.

The proof is easy.

For

$$AB^2 = OB^2 - OA^2$$

$$= \left(\frac{v^2}{g}+a\right)^2 - a^2 = \left(\frac{V^2}{g}+2a\right)$$

$$AB = v\sqrt{v^2+2ga}/g$$

$$= l, \text{ from Equation (6C.6)}$$

and

$$\text{angle AOB} = \tan^{-1}(AB/OA)$$

$$= \tan^{-1}(l/a)$$

$$= 20 \text{ from equation (6C.11).}$$

(b) *Solution for discharge co-efficient, C Values of l and a known from observation.*

The object of this is to estimate the discharge coefficient of the siphon, from the observed value of l. The steps are, Figure 6 C.20 :

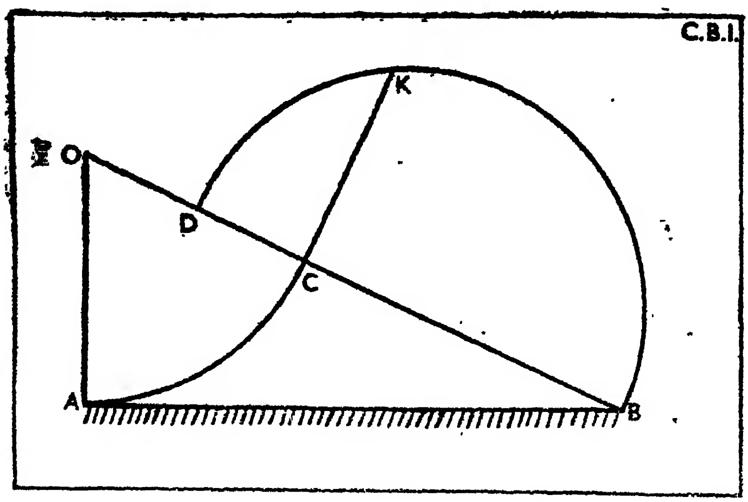

Figure 6 C.20. :—*Showing geometrical construction for discharge co-efficient.*

(*i*) Let 0 be the nozzle, A being the point vertically below it on the river bed, and AB the observed 'l'.

(*ii*) Join OB. Cut off OC=OA and then CD along BC=$h/2$ (where h is the depth of the nozzle below the crest).

(*iii*) Draw a semicircle on BD as diameter. Draw CK perpendicular to BC to meet the semicircle in K.

(*iv*) Then CK/h gives the coefficient. For proof,

$$OB^2 = OA^2 + AB^2 = a^2 + l^2$$

$$= a^2 + \frac{V^2(V^2+2ga)}{g^2} \quad \text{from Equation} \quad (6C.6)$$

$$\therefore \; OB = a + \frac{V^2}{g}$$

$$\text{so} \quad BC = OB - OC = a + \frac{V^2}{g} - a = \frac{V^2}{g}$$

Now $\quad CD \quad = h/z$, by construction.

$$CK^2 = BC.CD$$

$$= \frac{(V^2)}{g} \frac{1}{2}$$

or $$CK = V.\sqrt{h/2g}$$

$$CK = V\sqrt{2gh}$$

C, the discharge coefficient.

The dimensional aspect of the problem is brought into focus by re-writing Equations (6C.6) and (6C.11), after putting $v^2/ga=F$ (the Froude Number), in the form

$$l/a = \sqrt{F(F+2)} \quad \ldots \quad \ldots \quad \ldots \quad (6C.18)$$

and $$\theta = \tfrac{1}{2} \ \mathrm{Sec}^{-1} \ (F+1) \quad \ldots \quad \ldots \quad \ldots \quad (6C.19)$$

These non-dimensional equations indicate that in model experiments needed for estimating the prototype values of l/a and θ, the gravity forces should dominate those due to viscosity, surface tension *etc.*, and the ratio given most attention should be the Froude Number.

Now $F=v^2/ga=2C^2h/a$, where h is the depth of the nozzle below the crest, C the discharge coefficient and a the height of the nozzle above the river bed. Usually C will lie between 0·60 and 0·85, and so F may be expected to lie between 0·75 h/a and 0·85 h/a.

There is no particularly fixed range for the values of h/a; but the lower the orifice, the greater the jet velocity and the greater the distance to which water can be thrown. Too low an orifice, however, is also not desirable, as the extra siphon length means extra cost, and the higher velocity might damage the surface.

Perhaps h would ordinarily lie between 1/4 to 3/4 the height of the dam, so that h/a will vary between 1/3 and 3. The range for F may, therefore, be put as going from 0·25 to 4·50.

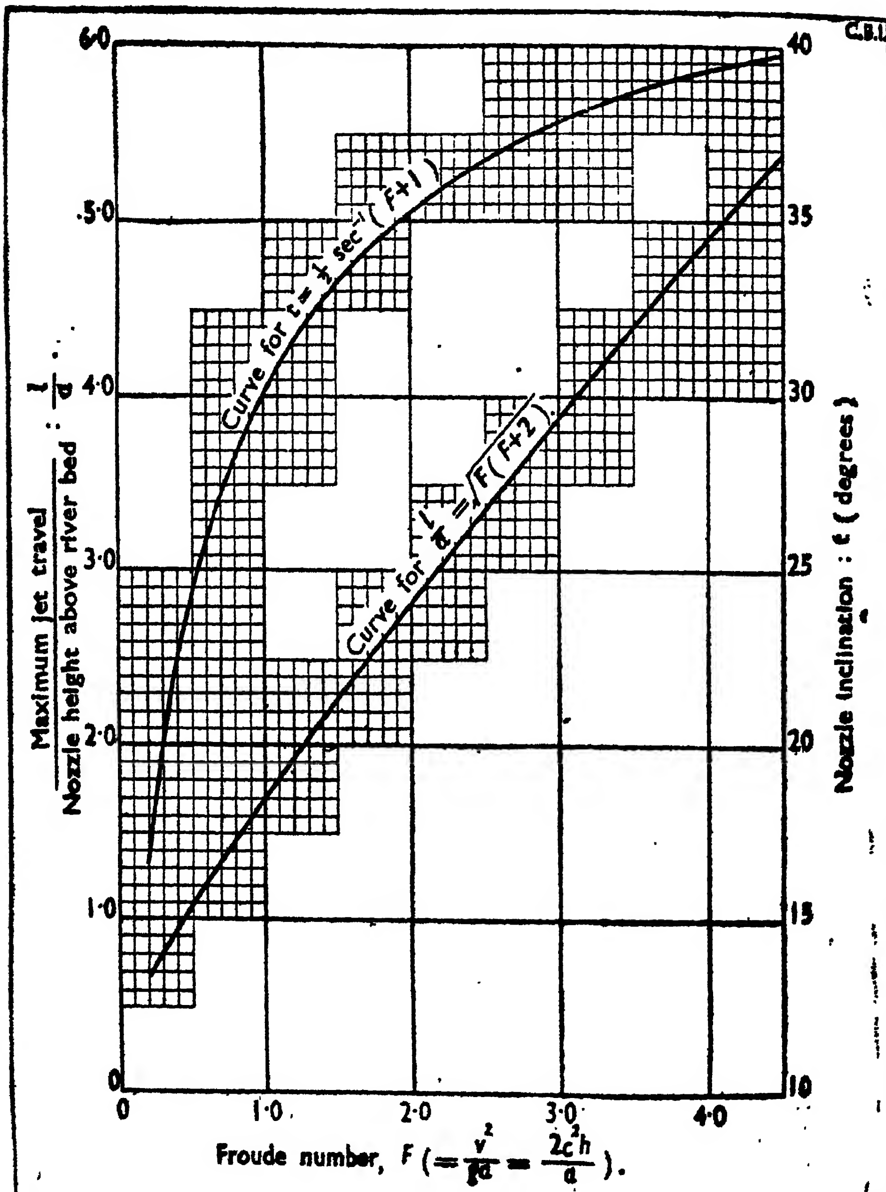

Figure 6 C.21: *Curves giving values of $\frac{l}{a}$ and t of values of F ranging from 0·2 to 4·5.*

The values of l/a and θ for this range of F can be easily read off the curves in Figure 6 C.21.

(16) FURTHER EXPERIMENTS ON THE DISSIPATION OF ENERGY OF FLOW FROM SIPHON OUTLETS [18]

Experiments were continued for dissipating the energy of jets from siphon outlets. They comprised of the following :—

(1) Supporting the jets on an ogee profile

(2) Submersion of outlet

(3) Formation of a standing wave

SUPPORTING THE JETS

A battery of six siphons operating under a head of 3·75 feet was chosen for the purpose. The under surface of the jet was traced and a slightly flatter parabolic curve was proposed for the supporting profile. The idea was that all the jets should fan out and reach the tail water in a uniform sheet of water. The fanning, however, is effective, but it is seen that when the adjoining jets meet, they form subsidiary jets which cannot be prevented unless the siphon outlets are far apart. It is also observed that such a proposal involves a heavy addition of masonry to the dam.

SUBMERSION OF OUTLET

This method seems to be the best solution for preventing the scour downstream of siphons. Recent experiments have revealed that there are far-reaching advantages in such a design. It is not necessary that the outlet should be completely submerged—in fact, the conditions worsen if the outlet becomes more than submerged. It is advantageous to allow a slight openig at the top to start with. That helps the air inside the siphon to be expelled for a small flow of water over the siphon rim. The following results show the advantages of submerged outlets. The experiments were conducted on a 18 inch siphon model with an operating head of 7 feet—6¾ inches.

Clearance between downstream water level and crown of the outlet in inches	Priming depth in inches
½	½
1½	½
2	1
3	1½

It is seen that when the clearance between the tail water and the crown of outlet is ½ inch, the priming depth was only ½ inch while for a clear overfall discharge, the same was 3½ to 4 inches. As soon as the siphon primes, the tail

(18) Hydraulic Research Station, Krishnarajasagar, Annual Report, 1947, pages 26—29.

water level rises and the outlet gets completely submerged. The co-efficient of discharge in such a case is as high as 0·7 and is better than other methods like fixing, adjutages or impinging neighbouring jets *etc.*

FORMATION OF A STANDING WAVE

For the formation of a perfect standing wave it is necessary to obtain the correct depths that conduce to such a formation. First, it is necessary to keep the outlet at bed level itself so that the jet is supported on the bed of river and to create conditions on the bed that help the formation of a perfect jump.

Experiments were conducted on a 12 inches siphon model, with an operating head of three feet. The outlet was kept at bed level to obtain the maximum head. Again to improve the priming qualities and to gain further head, the outlets were made elliptical. To effect an initial water-seal, the top of outlet pipe was kept level the bottom tapering, linealy from the bend to the exit. The area of cross-section, however, remained the same throughout varying uniformly from circle to ellipse from the lower end of the bend to the exit.

To see that the discharge was free and a high co-efficient was realised, a standing wave was proposed to be formed at a convenient distance from the toe of the dam. This necessitated the adoption of levels as also friction blocks as shown in the drawing.

The proposal of sloping aprons and friction blocks have the following advantages :—

(*a*) The siphon primes easily, the priming depth being 4½ inches to 5 inches.

(*b*) The water jet spreads out thus reducing the detrimental effect on the bed.

(*c*) The initial water-seal helps easy priming.

(*d*) A standing wave will be formed at a distance of about 1½ feet from the toe of the dam. The friction blocks help the formation of jump.

(*e*) The co-efficient realised is 0·735.

An improvement made on this is to change the circular section to elliptical not linealy as mentioned above, but parabolically. Thus a kink at the lower end is avoided and it will naturally improve the co-efficient of discharge. In the experiment conducted, the co-efficient was increased to 0·75.

(17) DESIGN OF VOLUTES *ETC.* IN SIPHONS [19]

For an effective design of the components of the volute siphons, the following factors should be taken into consideration :—

(1) A water-seal should be secured in the barrel.

(2) A vortex of sufficient strength should be ensured in order to expel all the air in the dome early.

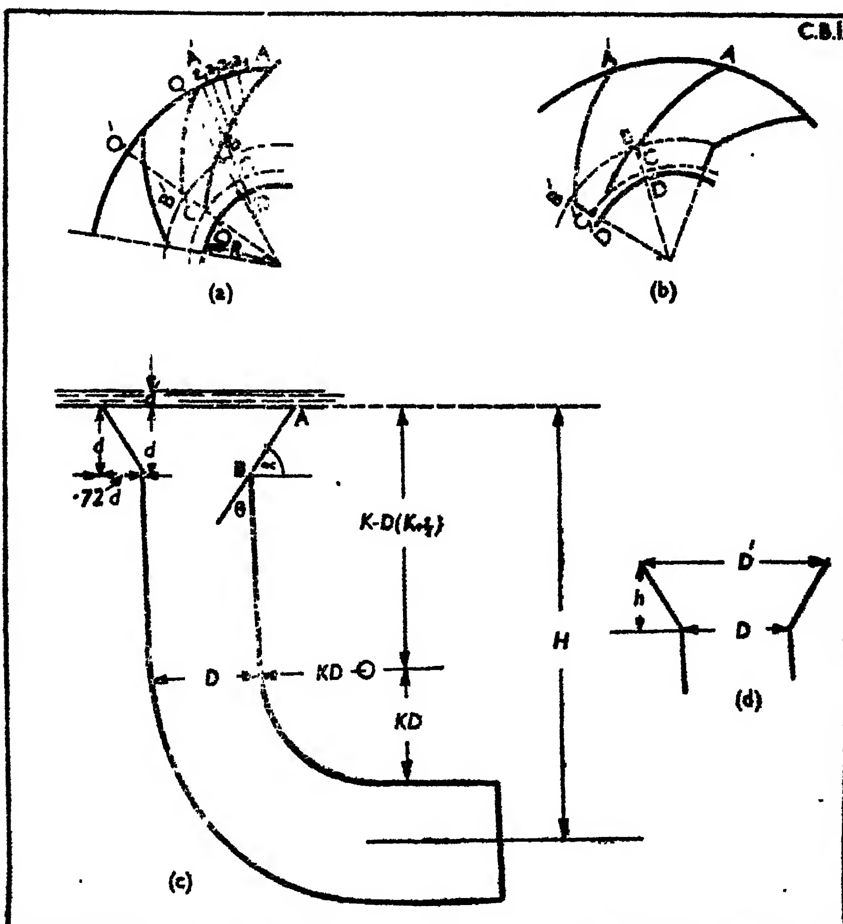

Figure 6 C.22

[19] Hydraulic Research Station, Krishnarajasagar, Annual Report, 1947, pages 20 to 42.

RELATIONSHIP BETWEEN THE BARREL DIAMETER AND THE HEAD

The assumption is that the boil should be formed in the barrel itself and in the limit, at the top of the bend, Figure 6 C.22 (a, b, c, d)

Let the slope of the funnel be 1 vertical in n horizontal.

Diameter of siphon barrel,	D
Height of funnel,	d
Priming depth,	d'
Radius of inner bend,	KD
Height of lip above centre of outlet,	H
Co-efficient of velocity.	C_v

Then the condition for the boil to form at the top of the bend is deduced as follows :—

Velocity at the throat$=C_v \times 8\sqrt{(d+d')}$

Angle of throw with horizontal-$\tan^{-1} \dfrac{1}{n}$

Using the usual projectile formula,

$$y=x \tan a+\frac{g}{2} \times \frac{x^2}{v^2 \cos^2 a}$$

and substituting $x=\dfrac{D}{2}$

$$y=\frac{D}{2} \cdot \frac{1}{n}+\frac{g}{2} \cdot \frac{D^2\left(1+\frac{1}{n^2}\right)}{4 \times 64\,(d+d')\,Cv^2}$$

But $y=H-D(K+\frac{1}{2})-d$.

$$\therefore\ H-D\,(K+\tfrac{1}{2})-d \leq \frac{D}{2} \cdot \frac{1}{n}+\frac{D^2\left(1+\frac{1}{n^2}\right)}{16\,Cv^2\,(d+d')}$$

$$i.e.,\ H>D\ \ \frac{1}{2n}+K+\tfrac{1}{2}\ +\frac{D^2\left(1+\frac{1}{n^2}\right)}{16\,Cv^2\,(d+d')}+d$$

Assuming, for Hirebhasgar Siphon.

$n=1$; $K=1$; $d'=\dfrac{D}{15}$, $Cv=0\cdot6$, and $d=\dfrac{D}{4}$, we deduce

$$H \leq 3\cdot1\,D$$

This condition is satisfied by the executed design.

FUNNEL

The design of the funnel should be such as to induce a high " boil point " without increasing frictional losses.

By increasing the funnel height h Figure 6 C.22(c) the initial velocity of the water leaving the throat at B will be large which is helpful in inducing the necessary circumferential velocity for the water-seal formed by the flow between the volutes. A high initial velocity increases twisting motion and, therefore, better sucking of air which facilitates early priming. A flat slope that is a lesser angle, combined with a high velocity induces the formation of a high boil point.

It is necessary the boil-point is formed within the vertical pipe or barrel portion and that as high as possible.

OPTIMUM FUNNEL SLOPE FOR LEAST TIME FOR FORMATION OF A " BOIL "

Let D' and D be the diameters of the funnel at lip and throat of a funnel.

The lip diameter is more or less fixed by the amount of discharge that has to be passed per foot length of spillway.

The throat diameter is fixed by the priming depth required as well as by the discharge to be allowed in each siphon.

Let us assume D' and D to be fixed.

Let α be the angle the funnel makes with the horizontal.

The problem is to determine the funnel slope so that the jet shooting from funnel bottom reaches the centre of barrel in the earliest possible time.

We have the horizontal distance to the centre of barrel$=\frac{D}{2}$

Height of funnel$=h$.

Let V be the velocity of water at the throat B.

Let t=time taken for the water to traverse the horizontal distance from from B to C the centre of barrel, that is a distance of $\frac{D}{2}$.

$g=32 \cdot 4$ feet per second.

y=height of boil below throat B.

Horizontal Component.

$$V \cos a\, t = \frac{D}{2} \qquad (6C.20)$$

But $$V = \sqrt{2gh} = 8\sqrt{\frac{D'-D}{2}\tan a} \qquad (6C.21)$$

Substituting in (6C.20)

$$\frac{D}{2} = \left\{ 8\sqrt{\frac{D'-D}{2}\tan a} \right\} \cos a\, t.$$

$$= \left\{ 8\sqrt{\frac{D'-D}{2}\tan a \cos^2 a} \right\} t.$$

$$= \left\{ 8\sqrt{\frac{D'-D}{4}\sin 2a} \right\} t.$$

Hence, $$t = D\,\frac{2}{8\sqrt{(D'-D)\sin 2a}} = \frac{D}{8\sqrt{(D'-D)\sin 2a}}$$

Putting, $$\frac{D}{8\sqrt{(D'-D)}} = K$$

we have $$t = \frac{K}{\sqrt{\sin 2a}}.$$

Differentiating with respect to a and equating to zero,

$$\frac{dt}{da} = \frac{-K.\cos 2a}{(\sin 2a)^{3/2}} = 0$$

i. e., $$\frac{\cos 2a}{(\sin 2a)^{3/2}} = 0$$

i. e., $\cos 2a = 0$ i. e., $a = 45°$.

Applying the usual projectile formula,

$$y=x\tan a+\frac{g}{2}\cdot\frac{x^2}{V^2\cos^2 a},$$

and substituting, $x=\dfrac{D}{2}$ and $a=45°$,

we have,

$$y=\frac{D}{2}+\frac{8D^2}{V^2}\quad\ldots\ldots\ldots\quad(6C.22)$$

$$\text{But } V^2=2g\,\frac{D'-D}{2}=g(D'-D)$$

$$\therefore\quad y=\frac{D}{2}+\frac{D^2}{4(D'-D)}\quad\ldots\ldots\ldots\quad(6C.23)$$

OPTIMUM SLOPE FOR HIGHEST BOIL CONDITIONS FOR GIVEN DIAMETERS AT THE LIP AND THE THROAT OF A FUNNEL

Let y be the depth of boil below throat B.

x, the distance of boil point from the barrel sides equal to $\dfrac{D}{2}$ in this case.

We have, assuming the projectile formula

$$y=x\tan a+\frac{g}{2}\cdot\frac{x^2}{V^2\cos^2 a}.$$

$$x=\frac{D}{2};\qquad V=8\sqrt{\frac{D'-D}{2}\tan a}$$

$$\text{Therefore}\quad y=\frac{D}{2}\tan a+\frac{g}{2}\cdot\frac{D^2}{4}\cdot\frac{1}{64\left(\dfrac{D'-D}{2}\right)\tan a\cos^2 a}$$

$$=\frac{D}{2}\tan a+\frac{D^2}{8(D'-D)\tan a\cos^2 a}\quad\ldots\quad(6C.24)$$

$$\text{Putting}\quad\frac{D^2}{8(D'-D)}=K,$$

$$y=\frac{D}{2}\tan a+K(\cot a\sec^2 a)$$

$$=\frac{D}{2}\tan a+K(\cot a+\tan a)$$

Differentiating with respect to a

$$\frac{dy}{da}=\frac{D}{2}\sec^2 a+K(-\operatorname{cosec}^2 a+\sec^2 a).$$

This must be zero for highest boil,

$$\therefore \quad \frac{D}{2}\cdot\frac{1}{\cos^2 a}+K\left(-\frac{1}{\sin^2 a}+\frac{1}{\cos^2 a}\right)=0$$

$$i.\,e.,\ \frac{D}{2}\sin^2 a+K\left(-\cos^2 a+\sin^2 a\right)=0$$

$$i.\,e.,\ \sin^2 a\left(\frac{D}{2}+K\right)=K\left(1-\sin^2 a\right)$$

$$i.\,e.,\ \sin^2 a\left(\frac{D}{2}+2K\right)=K \qquad i.\,e.,\ \sin^2 a=\frac{2K}{D+4K}$$

$$\text{or } a=\sin^{-1}\sqrt{\frac{2K}{D+4K}} \quad \text{where } K=\frac{D^2}{8\,(D'-D)} \qquad (6C.25)$$

substituting values for a and V in Equation 5,

$$y=\frac{D}{2}\sqrt{\frac{2D}{8D_1-7D}}+\frac{D\,(2D_1D-D^2)}{4\,(D_1-D)\sqrt{4DD_1-3D^2}} \qquad (6C.26)$$

The difference given in the height h of boil-point by the application of the Equations (6C.23) and (6C. 26) is not very great. In both the formulae, the diameters of the funnel at the lip and the throat are taken as fixed while the height and slope vary. In Equation (6C.23) the least time for boil-point formation is obtained by having a higher vertical component of the velocity at the throat B. While in Equation (6C.26) the highest boil-point has been obtained by having a larger horizontal component of the velocity at the point of throw-off at the throat B. The height of funnel required is lesser and the slope flatter than in the former case. By designing the funnel according to Equation (6C.22), the volume of air to be sucked out will be least but the velocity of the water issuing out from the throat B will be smaller than in the former case on account of the lower height of the funnel. The momentum of the water thrown off from the throat may be too small to resist the puncturing action of the air rushing in from the outlet to break the pressure difference as soon as it is created. With a lower velocity, a larger volume of water will be required to gain the necessary momentum to form an effective water-seal. That means, the priming depth will have to be high.

If it is designed according to Equation (6C.23) the volume of air to be sucked out will no doubt be more, but on account of the bigger height of the funnel the water at the throat B is thrown out with a larger initial momentum, resulting in a more powerful sucking force. A bigger height is favourable also for volutes to come into full action. The priming depth in consequence may become lesser. If the difference in volume as measured from the crown of the dome to the boil-point is not very great between the two designs, it is better to choose the steeper and higher funnel.

The choice of the slope and height of funnel is thus largely a matter of judgement based on experience. A lip 1·6 times the diameter of the barrel with a funnel height equal to one third of the diameter of the barrel will generally answer the purpose. This gives a funnel slope of 1 to 1.

The diameter of the funnel at the lip has been deliberately assumed as fixed. An increase in diameter will not only lessen the discharging capacity of the design per foot length of spillway available but also increases considerably the cost of construction out of all proportion to the resulting small decrease in priming depth.

Other devices for forming a high water-seal have been tried experimentally. One was to shape the funnel like a bowl. While this no doubt aids the formation of a high water-seal, the momentum of the water thrown off from the throat does not in any way get increased. There is also the danger of cavitation occurring at all the depressed portions when the siphon is discharging full. The flow from the lip overrides the volute ribs producing turbulent flow, thus reducing the co-efficient of discharge. Instead, two slopes, one of 1½ to 1 for two thirds of the height of the funnel measured from the lip level and another of 1 to 1 for the remaining one third height was next tried. A small shoulder at the junction of the two slopes was also introduced. Since this had no significant effect on either priming or discharge this was dropped. Instead, the two slopes were joined by a smooth curve. This also was found to have the same disadvantages as that of a bowl-shaped one, instead of any added advantage.

The lip of the funnel should be round and smooth to allow easy access for water.

The throat, that is, the junction line between the funnel and the vertical pipe should be absolutely sharp. It is better the throat end is just a fraction of an inch projecting into the vertical pipe diameter, so that every drop of water flowing along the funnel is thrown out, and aids in the formation of a water-seal. Any attempt at bell-mouthing the junction of the throat with the barrel will result in water hugging to the sides, increasing priming depth considerably.

VOLUTES

The performance of a siphon depends to a large extent on the design of the funnel and the volutes.

As explained below the design of the funnel and the volutes should be such as to cause priming, by first creating a high water-seal and then evacuating air (*a*) by dragging it at the boundary layer due to velocity of water, (*b*) by screwing it out by giving a spiral motion to the water-seal, (*c*) by creating vapour particles in the air space above boil point and (*d*) by physically trapping it.

The design of the funnel governs the position of the water-seal.

The design of the volutes governs the methods of evacuating air.

This depends on the number, height, shape and orientation of the volutes.

ORIENTATION OF THE VOLUTES

Let ABC and A′ B′ C′ be two adjoining volutes, Figure 6 C.22 (a) of a siphon with n volutes.

Let D D′ be the throat of the funnel which is also the radius of the barrel.

Let O B C Q and O B′ C Q′ be two radii such that the arc Q Q′ is 1/24th the circumferential length. Let O D=R. Leave a small margin say equal to 3/10 R from the throat and draw another circle which will cut O D Q and O D′ Q′ at C and C′ respectively.

S is the sloping length of the funnel. Along the radial lengths C Q and C′ Q′ measure CB=C′B′=p ft. join C′ B′ by a line and produce it to meet the lip at A′. Then A′ B′ C′ is one volute.

Similary draw the other volutes A B C *etc.*

Let n be the number of volutes.

Then $\frac{2\pi}{n}$ is the angle subtended at the centre by any pair of volutes.

Slope of funnel=1 to 1.

Let θ be the angle the filament flowing at B makes with the radial length BC, the angle being measured in a plane containing BC and parallel to the funnel plane at BC.

Let p=length BC of the radial portion of the volute
h=hight of funnel.

The water enters the lip radially in filaments $q_1\, q_1'$, $q_2\, q_2'$, $q_3\, q_3'$, *etc.*, and meets the volute AB. All the filament paths get deflected along the volute, and gain a certain inertia of movement in the direction A B as they flow hugging the volute face A B. At B, they have to leave the volute. The paths of the filament now get deflected again due to the slope of the funnel. The orientation AB of the volute should be such that when these filaments fan out in the portion BC′C, one extreme filament from AB should strike the tip C′ of the next volute, and the other extreme filament should flow radially along the path BC of the volute. That is, the circumferential velocity for one extreme filament should be zero, so that its path is not deflected from the radial line BC, and the circumferential velocity for the other filaments should go on increasing till it is sufficient to make the other filament BC′ strike the tip of the neighbouring volute at C′. In such a design, the water will fan out uniformly over the area BC′C.

Velocity acquired by water filaments at B

$$=\sqrt{2g}\cdot\sqrt{h-\frac{P}{\sqrt{2}}}$$

The vertical distance to be travelled between B and C$' = \frac{P}{\sqrt{2}}$

Horizontal distance $=2\ R\sin\frac{\pi}{n}+\frac{P}{\sqrt{2}}\sin\frac{\pi}{n}$

$$=\sin\frac{\pi}{n}\left(2R+\frac{P}{\sqrt{2}}\right)$$

Horizontal velocity component at $B=\sqrt{2g}\sqrt{h-\frac{P}{\sqrt{2}}}\cdot\sin\theta$

Vertical velocity component at $B=\sqrt{2g}\sqrt{h-\frac{P}{\sqrt{2}}}\cdot\frac{\cos\theta}{\sqrt{2}}$

We have for the horizontal and vertical distances travelled in time t,

$$\sin\frac{\pi}{n}\left(2R+\frac{P}{\sqrt{2}}\right)=\sqrt{2g}\sqrt{h-\frac{P}{\sqrt{2}}}\cdot\sin\theta\ t. \qquad \text{....(6C.27)}$$

$$\frac{P}{\sqrt{2}}=\sqrt{2g}\sqrt{h-\frac{P}{\sqrt{2}}}\cdot\frac{\cos\theta}{\sqrt{2}}\ t+\tfrac{1}{2}gt^2 \qquad \text{....(6C.28)}$$

From (6C.27) we have

$$t=\frac{\sin\frac{\pi}{n}\left(2R+\frac{P}{\sqrt{2}}\right)}{\sqrt{2g}\sqrt{h-\frac{P}{\sqrt{2}}}\cdot\sin\theta} \qquad \text{....(6C.29)}$$

Substituting in (6C.28)

$$\frac{P}{\sqrt{2}}=\sin\frac{\pi}{n}\left(2R+\frac{P}{\sqrt{2}}\right)\frac{\cot\theta}{\sqrt{2}}+\frac{g}{2}\cdot\frac{\sin^2\frac{\pi}{n}\left(2R+\frac{P}{\sqrt{2}}\right)^2}{2g\left(h-\frac{P}{\sqrt{2}}\right)\sin^2\theta}$$

That is

$$p=\sin\frac{\pi}{n}\left(2R+\frac{P}{\sqrt{2}}\right)\cot\theta+\frac{\sqrt{2}}{4}\cdot\frac{\sin^2\frac{\pi}{n}\left(2R+\frac{P}{\sqrt{2}}\right)^2}{\left(h-\frac{P}{\sqrt{2}}\right)\sin^2\theta}$$

....(6C.30)

This gives the relationship between the radial length p and the slant length AB in a volute. It is seen that shorter the length of p, greater the circumferential velocity. The filaments under a high circumferential velocity from a powerful spiral vortex hugging the sides of the barrel with a very thin or no water-seal. The radial filaments will be too feeble to overcome the inertia set up by the circumferential velocity to meet in a point to form the boil. If the radial length p is made very large the circumferential velocity will be too small to induce proper fanning out or be helpful in creating a vacuum quickly. The efficiency of priming depends on the strength of the circumferential velocity. It should, therefore, be made as strong as possible without breaking water-seal formation.

Model Experiments show that $\frac{p}{\sqrt{2}}=\frac{h}{3}$ gives satisfactory results.

In a siphon, where

$n=24$, $R=18$ feet, $h=5\cdot75$ feet, we have $\theta=55^\circ$ applying the Equation (6C.30).

Actually there will be a volume of water flowing along the volute and not a single filament as assumed in deriving the Equation (6C.30). This induces friction and deviations in flow. It is not sufficient if the extreme filament first touches the tip C′ of the other volute. It must strike against the tip so as to induce formation of a ribbon-like jet. When these ribbon-like jets are thrown out from the various tips like C, these by throwing out spray when they meet one another help in sucking out air. A certain amount of air is also trapped between these filaments and those fanning out and forming a rotating water-seal. Actual experiments have shown that the single θ to be taken should be only nine-tenths of the calculated value for effective priming.

$Q=K\theta$ .... (6C.31)

Where Q is the angle CBC′ actually required θ is the angle calculated theoretically.

$K=0\cdot90$ $Q=0\cdot90$ .... (6C.32)

Variation of Velocity from C to C′.—Consider any point X on CC′. Let λ be the angle subtended at the centre by the chord CX. Since the length of arc CC′ is small, it may be taken as equal to the chord length CC′.

$$CX=2R\sin\frac{\lambda}{2}.\ \text{BC in the plane of the funnel}=p$$

$$\angle BCX=90^\circ+\frac{\lambda}{2}$$

$$BX^2=\left(2R\sin\frac{\lambda}{2}\right)^2+p^2-4Rp\sin\frac{\lambda}{2}\cos\angle BCX$$

$$=\left(2R\sin\frac{\lambda}{2}\right)^2+p^2+4Rp\sin\frac{\lambda}{2}\sin\frac{\lambda}{2},\ \text{since}\cos\angle BCX$$

$$=\cos\left(90+\frac{\lambda}{2}\right)=-\sin\frac{\lambda}{2}$$

$$\therefore BX^2=\sin^2\frac{\lambda}{2}(4R^2+4Rp)+p^2$$

$$\text{or } BX=\sqrt{\sin^2\frac{\lambda}{2}(4R^2+4Rp)+p^2} \qquad (6C.33)$$

Let θ_1=angle between BX and its projection on the horizontal plane,

$$\sin\theta_1=\frac{P}{\sqrt{2}}.BX \qquad (6C.34)$$

$$\cos\theta_1=\sqrt{1-p^2/2BX^2} \qquad (6C.35)$$

$$\sqrt{\frac{2BX^2-p^2}{2BX^2}}$$

$$=\sqrt{2}\sqrt{\sin^2\frac{\lambda}{2}(4R^2+4Rp)-p^2}$$

$$=\frac{}{\sqrt{2\sin^2/2\,(4R^2+4Rp)+p^2}} \qquad (6C.36)$$

Velocity of water at X= $\sqrt{2gh}$

Velocity in the direction of the projection of BX on the horizontal plane

$$=\sqrt{2gh}\cos\theta_1$$

Therefore the radial component at X in the horizontal plane
$=2gh\cos\theta_1\cos\angle OXB$ (OXB in the horizontal plane)
Circumferential component at X. (6 C.37)

$$=\sqrt{2gh}\cdot\cos\theta_1\cdot\sin\angle OXB. \qquad (6\ C.38)$$

Now in the triangle OBX in the horizontal plane

$$\frac{BX\cos\theta_1}{\sin\lambda}=\frac{OB}{\sin\angle OXB}$$

$$\text{Therefore } \sin\angle OXB=\frac{OB\sin\lambda}{BX\cos\theta_1}=\frac{\left(R+\frac{P}{\sqrt{2}}\right)\sin\lambda}{BX\cos\theta_1}$$

$$\cos\angle OXB=\frac{BX\cos\theta_1}{\sqrt{BX^2\cos^2\theta_1-\left(R+\frac{P}{\sqrt{2}}\right)^2\sin^2\lambda}} \qquad (6C.39)$$

Radial component at X in the horizontal plane, substituting for cos $\angle$OXB in Equation (6 C.39),

$$=\sqrt{2gh}\cdot\frac{\cos\theta_1\cdot BX\cos\theta_1}{\sqrt{BX^2\cos^2\theta_1-\left(R+\frac{P}{\sqrt{2}}\right)^2\sin^2\lambda}}$$

$$=\frac{\sqrt{2gh}\,BX\,.\cos^2\theta_1}{\sqrt{BX^2\cos^2\theta_1-\left(R+\frac{P}{\sqrt{2}}\right)^2\sin^2\lambda}} \qquad (6C.40)$$

The values of BX and cos λ are obtained from Equations (6C.33) and (6C.34) respectively.

Circumferential component at X, in the horizontal plane

$$=\sqrt{2gh}\quad\cos\theta_1\frac{\left(R+\frac{P}{\sqrt{2}}\right)\sin\lambda}{BX\cos\theta_1}.$$

$$=\sqrt{2gh}\,\frac{\left(R+\frac{P}{\sqrt{2}}\right)\sin\lambda}{BX} \qquad (6C.41)$$

The value of BX is obtained from Equation (6 C.33). Substituting this, we have

$$=\frac{\sqrt{2gh}\left(R+\frac{P}{\sqrt{}}\right)\sin\lambda}{\sqrt{\sin^2\frac{\lambda}{2}(4R^2+4Rp)+p^2}} \qquad (6C.42)$$

When $\lambda=0$, radial component $=\dfrac{\sqrt{2gh}.\ BX.\cos^2\theta}{BX\cos\theta_1}$

$$=\sqrt{2gh}.\cos\theta_1$$

Substituting $\lambda = 0$ in the Equation (6C.40).

This is the maximum value attainable of the radial component, and at this point, circumferential component$=0$, which is easily seen by substituting $\lambda=0$ in the Equation (6C.41).

Similary when $\lambda = \dfrac{2\pi}{n}$, the radial velocity is a minimum as can be seen from the same Equation (6C.40).

The circumferential velocity is then a maximum (Equation (6C.41). But the value of neither of the components is zero.

For effecting water-seal, the value of $\sqrt{2gh}\cos\theta_1$ should be such that with this velocity of throw off at the throat, all such filaments from the several volutes form a boil within the vertical pipe.

We have also seen the circumferential velocity varies from point to point. The velocities of all the filaments are the same at B. All the filaments have to run down the same height $\frac{p}{\sqrt{2}}$ before they leave the throat. But the distances traversed by the filaments vary from BC to BC′ or the angle with which the filaments are thrown off varies from

$$\sin^{-1}\frac{P/\sqrt{2}}{BC}\text{ to }\sin^{-1}\frac{P/\sqrt{2}}{BC''}.$$

Due to these different angles of throw off the filaments issuing forth are not normal to the circumference of the throat. The steepest filament is at C while the flatest is at C′. The result is, the fanned out layer of water occupying the space BCC′ is thrown out in a tilted plane. This happens in the space between the next pair of volutes as well. The filament near C′ in the next volute is the steepest, while the filament at C′ is the flatest. Due to sudden change in the angles of throw off between the two filaments, there will be a gap *which traps the air* between them.

We have thus a series of petal-like formations like those of a rose flower each petal overlapping the other. Due to the circumferential velocities in them they rotate closing the gap between the petal-like formations and entrapping and screwing in air. This is further assisted by the throw off of the ribbon-like water filaments issuing forth from the tip C′ by a certain portion of the flow between the volutes impinging against it. The radial velocities in the filaments also carry air with them in their boundary layers. Thus air is evacuated by all the four processes.

The volutes have to be as thin as possible so as not to lessen the area available for discharge at each section of the funnel diameter. They may be tapered both at the lip and the throat like the cut waters of bridge piers so as to offer minimum resistance to flow.

Owing to their thickness which is necessary for their stability, it is necessary to end them a distance of about $s/10$ above the funnel bottom to ensure a complete film of water all round the funnel at the throat. (s = the sloping length of the funnel).

The number and the height of funnel go together. The greater the number of volutes the lesser will be the priming depth upto a certain optimum limit with corresponding decrease in the co-efficient of discharge. The height of volutes should necessarily be not less than the expected priming depth, to prevent interference of flow between one pair of volutes and another. The reduction in priming depth is due to increased spiral movement caused when number of volutes is increased. We have already seen that the angle of these volutes can be so adjusted as to get a film of water covering the entire area between them. But greater the angle θ in Figure 6C.22 (a) greater will be the distances AA′ between the volutes. A very large increase will make all the water filaments hug the inner sides of the volutes so that the difference between the velocities between C and C′ will become very great causing unevenness of flow. The momentum of the filaments at C′ may become too low to resist the puncturing action of the air rushing in from the outlet whenever a difference of pressure is created above the water-seal portion. It is, therefore, desirable to have the volutes as close together as possible without unduly reducing the co-efficient of discharge. They may also be convenient multiples of four or six to ensure symmetry of flow.

Scale model experiments conducted at the Hydraulic Research Station on 12 in. model (that is, the diameter of the vertical pipe was 12 inches) operating under a head of 3·25 feet gave the following results :--

Experiment No.	1	2	3	4	5	6	7	8	9	10	11
Number of volutes.	6	6	6	9	9	9	9	12	12	12	12
Height of volutes.	¾	1	1½	¾	1	1¼	1½	¾	1	1¼	1½
Priming depth.	6	6	5½	5¼	5	5	4½	4½	4	3¼	3
Coefficient of discharge.	0·85	0·85	0·83	0·81	0·81	0·80	0·80	0·78	0·78	0·76	0·75.

All dimensions are in inches.

In small models the distribution of flow between volutes cannot be studied well. Any small error in the distribution, or turbulence caused by low volutes does not affect priming or discharge seriously. The results are indicative of the effect of increasing the number and height of volutes.

Regarding the section of the volute, its sides may be kept normal to the plane of the funnel with the top edges rounded off a little to give smoothness to the discharge when the funnel is flowing full after priming. Cup shapes and other shapes imitating the blades of a centrifugal pump or turbines have not been found to be advantageous as per experiments conducted at the research station.

The edge B of the volute ABC in Figure 6 C.22 (b) that is, the junction between the radial and inclined straight lengths of the volute may be smoothened with a small curve.

In order to bring about a uniform distribution of flow between the volutes at the throat, the effect of giving a small counter slope or superelevation from B towards B′ was tried. This causes turbulence in the flow and is not so good a device as the one obtained by suitable orientation of the volutes.

(18) EXPERIMENTS ON THE HIGH-HEAD SIPHON AT SHIMSHA [20]

A volute siphon of 10 inches diameter with an operating head of 62 feet has been put up in Handihalla between Shivasamudram and Shimsha. The siphon funnel and dome are enclosed in a steel tank which is fed with water from two cast iron pipes which in turn get their water from the penstock pipe carrying water to the Shimsha hydro-electric works.

The priming depth is very low (less than ½ inch). But there is so much turbulence in the tank that air is not prevented from entering inside. The outlet runs full and though an accurate determination of the discharge could not be made, the co-efficient of discharge was found to be very low being about 0·43. That is mainly due to two reasons, firstly entry of air into the dome by the turbulence of water in the tank, and secondly as the diameter of the siphon is too small compared with the head—which will not be the case in actual practice—the losses by friction are very great.

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(19) FURTHER EXPERIMENTS ON SIPHONS WITH VARIOUS FUNNELS AND OUTLETS [21]

With a view to improving the priming depth and as suggested by several engineers, several investigations were made with regard to

(1) Volutes

(2) Outlets

VOLUTES

In the light of the mathematical investigations experiments were made to test the design for the Jog siphons. Twenty-four volutes were proposed. Each volute would consist of two straight portions, one being radial and extending from the bottom to one third of the funnel in order to prevent any air gap that may be formed other wise at the tip of the volutes in the plane of the throat, and the other taking off from the radial end to the tip of the funnel at the required orientation, with the volute ends tapered. This method has been found effective. As deduced, the angle between the two straight portions of the volute should be 55° (exterior angle). Experiments showed that while this angle was true for a single filament touching the tip of the next volute, 45° angle gave more uniform distribution of flow. The corner at the junction of the radial and

[20] Hydraulic Research Station, Krishnarajasagar, Annual Report, 1947, page 43.

[21] Hydraulic Research Station, Krishnarajasagar, Annual Report, 1947, Pages 43-47.

oriented length of the volute was rounded off by a 1 foot radius curve. This ensured an even distribution of water along the length of the throat, at the same time giving the desired effects of a water-seal and circumferential velocity to water filaments. The thickness of volutes were made 6 inches and the height 1·5 feet from the bend to half the upper limb and tapered off towards both ends to a height one foot (Figure 6 C.23).

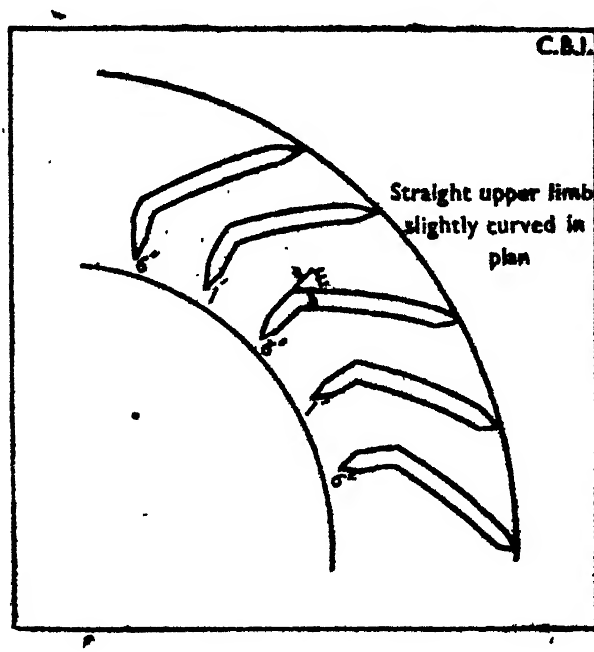

Figure 6 C.23

This proposal was verified with a three feet diameter vertical barrel with a funnel of slope 1 : 1 of a height of one foot which corresponded to a 1/8 scale model of Hirebhasgar siphons. No dome was used, nor any bend and the necessary outlet. The funnel was mounted on a three feet diameter circular well and the water discharging into this well was conveyed outside by means of a two feet square outlet. The experiments indicated the nature of the streamlines through volutes before priming and also they indicated how the design brought about the water-seal and twisting effects. While there was a uniform filament of water all round the throat both having a twisted motion and creating a water-seal, some filaments of water striking the noses of the volutes were deflected and thrown toward the centre, the whole thing appearing as a set of petals of a

flower and behaving as a device to entrap and suck the air out effectively Photograph Figure (6 C.24) indicates the stream lines. The petal formation superposes the uniform flow over the throat.

Figure 6 C.24

EXTERNAL VOLUTES

It was suggested that an alternative design on the principle of a turbine could be usefully investigated. Experiments were conducted on a one foot diameter siphon. The barrel has no funnel at the top. Water is guided into the vertical barrel through a set of volutes mounted on an external inverted funnel (Figure 6 C.25)—all other things remaining constant. This kind of siphon primed early for a depth of 1½ to 2 inches. The effect of the volutes before priming is very little and after priming it induces terrible vibrations in the dome. This is due to a greater area of high velocity region between the dome and the

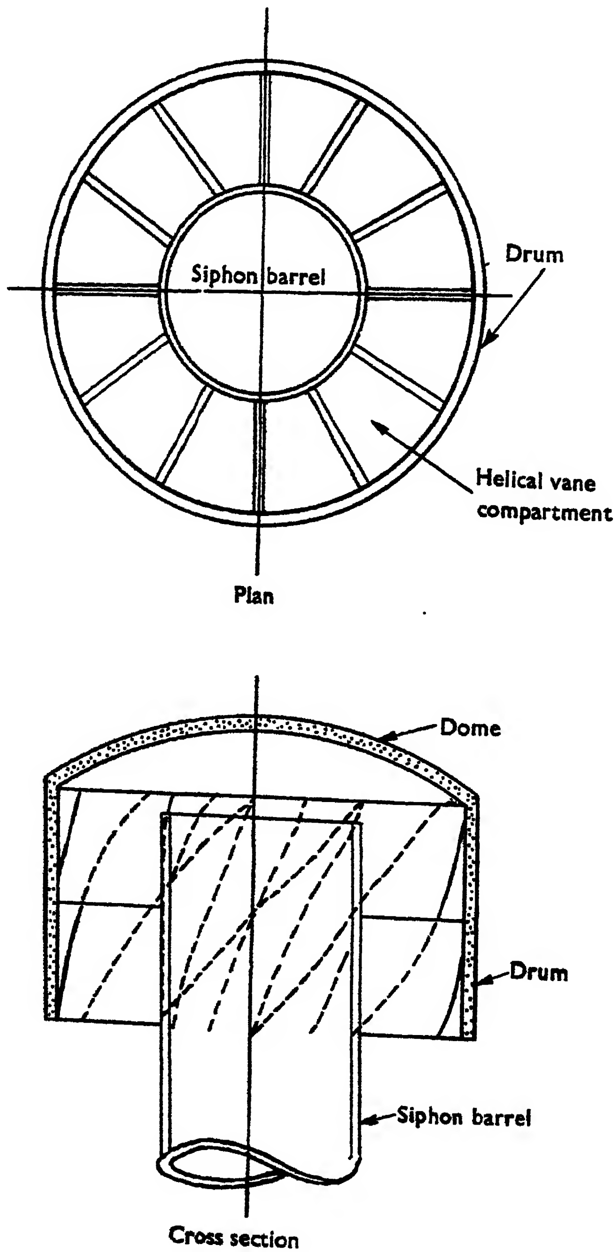

Figure 6C.26:- *Showing detailed drawing of turbine siphon*

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barrel. The co-efficient of discharge is necessarily low owing to great losses at the top of the barrel and is found to be 0·65. In order to lessen the vibration, the dome was raised by two inches and it was found that the vibration decreased enormously.

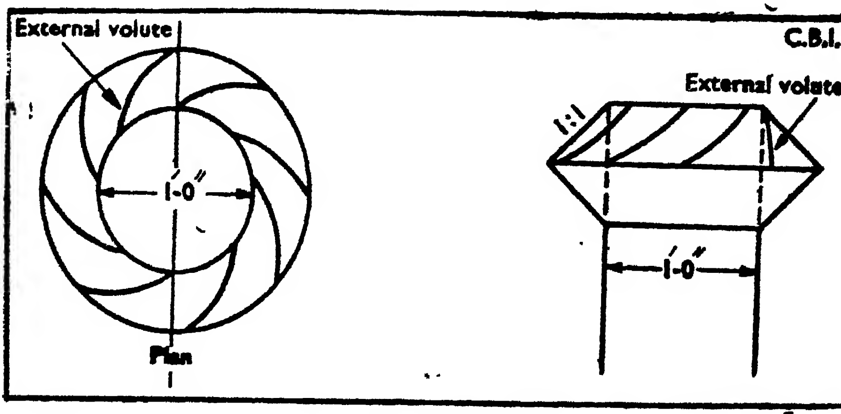

Figure 6 C.25

TURBINE SIPHON

Yet another suggestion was to join the drum and the vertical barrel by means of helical vanes. This would result in separate compartments which should guide the water into the vertical barrel in an angular direction. This was tried in a one foot diameter siphon (Figure 6 C.26). It was found that even two feet of depth of water above the lip could not prime the siphon. A huge air core was formed at the top of the dome and its diameter diminished with the increase of depth but never completely.

SIPHONS WITH CLOCKWISE VOLUTES

To observe if the change in the direction of the volutes would improve the priming depth, all the volutes in a one foot model were made clockwise. The priming depth did not improve.

VOLUTE SIPHON WITH RECTANGULAR OUTLET

As a measure of reducing the priming depth it was suggested to make the outlet rectangular. The change was made from circular at the top of the bend to rectangular at the lower end of the same area and width of outlet being as the diameter. In the experiments made, the siphon barrel was one foot in diameter, the outlet $1' \times \frac{\pi'}{4}$. This model with a head of 3½ feet primed for a depth of 4½ inches and ran full bore. The priming depth is high because, before priming water flows to a very small depth at the outlet.

In the light of these experiments the following deductions may be made :—

(1) A greater number of thin volutes mathematically derived are more helpful than a smaller number of thick ones widely spaced.

(2) To put volutes on the outside of barrel does not as such help priming but causes serious vibration in the dome after priming.

(3) Joining the drum and barrel by guide vanes and thus forming separate helical compartments prevents priming at reasonable depths. This indicates that radial velocity is very necessary.

(4) To make the outlet rectangular instead of circular does not help in improving priming depth.

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(20) DESIGN OF A HIGH CO-EFFICIENT WEIR AND SCOUR PREVENTER FOR THE TUNGA ANICUT [22]

ABSTRACT

The subject matter for this article has been the result of numerous experiments conducted at the Hydraulic Research Station, Krishnarajasagar, by way of investigating into the design of a suitable high co-efficient weir spillway for the Tunga River at Sakrebyle. The normal width of the River at site is about 1,000 feet and the computed discharge 250,000 cusecs. Assuming the ordinary trapezidal section for the spillway, it requires 1,700 feet length, with a spillage depth of 13·75 feet. This entails the widening of the river and cutting a draft channel at the site.

Investigations were made with the help of small models in order to design a high co-efficient weir the idea being either ;

(1) To effectively cut down the length of spillway, or

(2) To lessen the depth of spillage and hence the area of land to be acquired, or

(3) To increase the storage in order to supply water to sugarcane crops in the dry season.

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[22] Hydraulic Research Station, Kishnarajasagar, Annual Report, 1947 pages 47—67.

THEORY

The usual weir formula is :

$$Q=C.L.\ (h+h_a)^{1.5}-h_a^{1.5}$$ where

Q=Dishcarge in cusecs,

C=Co-efficient of discharge, 3·1 for broad crested weirs and 3·33 for sharp crested weirs (Francis's co-efficeint)

L=Length of weir

h=Head measured over crest

h_a=Head due to velocity of approach

When good stilling arrangements are made in experimental units the velocity of approach is very small compared to the actual head, so that the formula is written in a more convenient form

$$Q=CL\ (H+h^a)^{1.5}$$

It is possible to improve the value of C by suitably designing the profile. The usual method of doing it is to design it as the undernappe of the sheet of water falling clear from a sharp-crested weir (Figure 6 C. 27).

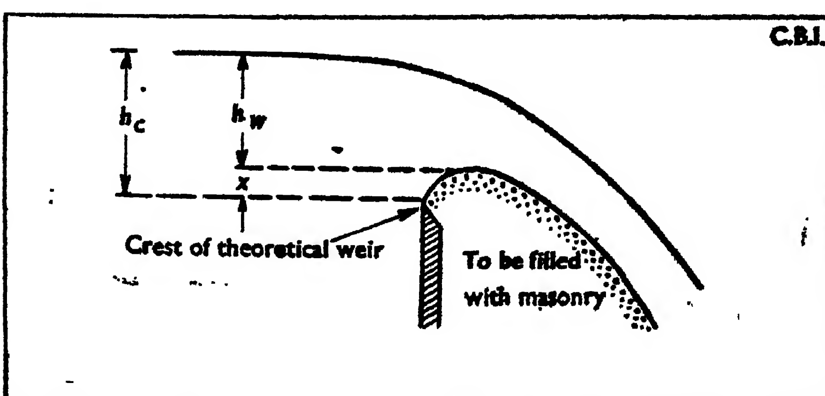

Figure 6 C.27

Let h_w=head over the crest of weir,

h_c=depth of water surface over the theoretical crest of sharp-crested weir,

$x=h_c-h_w$.

Then experiments show that $X=0\cdot126\ h_w$.

To arrive at the theoretical coefficient of such a weir the Francis's co-efficient of 3·33 must be increased accordingly.

Now we have,

$$Q = C.L.\ h_w^{1.5}$$

$$= C.L.\ (1{\cdot}126\ e_w)^{1.5}$$

$$= 3{\cdot}33 \times 1{\cdot}126^{1.5} \times L \times h_w^{1.5}$$

$$= 3{\cdot}98\ L.\ h_w^{1.5}$$

In a weir so designed, the nappe just adheres to the surface of the profile and the pressure at all points on it will be atmospheric.

To increase the coefficient of discharge still further, the profile may be designed for a head smaller than the actual value. In such a case, due to the suppression of the nappe, the curvature of stream lines at crest will be more, with a consequent steepness in the pressure gradient and negative pressures develop and the co-efficient will increase.

The theoretical profile is difficult to be adopted in practice. So alternatives having all the required qualities are to be sought for. Moreover, unless the height of the spillway dam is high (more than 50 feet) the sections so designed will be more than necessary for stability.

CO-EFFICIENT OF DISCHARGE

The co-efficient of discharge depends on various factors, all of them helping to increase the curvature of stream lines at the crest. The following may be mentioned :

(1) The nature of the profile at the top.

(2) The slope of the rear glacis.

(3) The $\frac{D}{H}$ ratio.

(4) The approach conditions which include silt accumulation in front of the weir.

(5) The number of end contractions.

(6) The extent of the negative pressure developed at the crest.

Schoklitsch has stated that the silt accumulation in front of weirs helps to increase the co-efficient. In the usual models the increase could not be discerned and extensive experiments are necessary for establishing the nature of silt and the nature of formation that effects such an increase.

End contractions have the effect of a reduced weir length and hence the co-efficient will fall. So the number of end contractions should be minimised.

The extent of the negative pressure developed at the crest depends on the top curve designed and the two together cause a sharper curvature of stream lines at the top. The absolute value depends on the spillage depth. The negative pressure should be well within the cavitation limit *i.e.*, within about 24 feet of water.

It is found that the co-efficient depends on the head-height ratio or the $\frac{D}{H}$ ratio. Up to a limit the co-efficient increases with the ratio and then falls again. The peak point depends on the profile in question and unless the corresponding head is reached it cannot be realised.

Thus it becomes necessary to choose such a profile, from a $\frac{D}{H}$ —Cd curve which has a peak point as near as possible to actual critical conditions of discharge.

EXPERIMENTAL PROCEDURE

For an accurate determination of the co-efficient of discharge in high co-efficient weir models, the following factors are essential :

(1) *Measurement of velocity of approach.*—In all experiments, the velocity of approach cannot be prevented, in spite of elaborate stilling arrangements. It becomes necessary to add the corresponding head to the head read by means of gauges. The head due to approach velocity is given by $h_a = \frac{V}{2g}$ where V=the effective velocity.

The effective velocity depends on the nature of the distribution of velocity in the cross-section. It is not constant for two units, nor is it so for all flow conditions.

It is possible to determine the mean velocity by measuring the discharge and dividing it by the cross-section. This must be multiplied by a co-efficient a to get the effective velocity.

$V_{av} = \frac{Q}{A}$ and the head required may be expressed as

$$H_a = a\frac{V_{av}}{2g}$$

Where a is a correcting co-efficient which is always greater than unity and within about 1·3.

The actual determination of the corrective co-efficient is rather difficult. The convenient way of doing it is to construct a sharp crested weir of the same

height as that of the model and comparing the usual weir formula and the Rehbock formula for the discharge. Rehbock formula for discharge over sharp-crested weirs, with end-contractions suppressed is

$$Q=2/3\sqrt{2g}LH^{3/2}\left(0{\cdot}605+\frac{1}{320\,H-3}+\frac{0{\cdot}08H}{P}\right)$$

where H=depth of spillage

P=height of weir

The usual formula is

$$Q=CL\,(H+h_a)^{3/2}$$

when the right hand functions are equated the value of h_a is deduced.

Then, $a=\dfrac{2\,g{\cdot}h_a}{V_a^2}$

The values of a were determined for a low value and a high value of H and the intermediate ones were interpolated by proportion.

(2) *Model Technique.*—It is necessary that the model should behave similar to the prototype. The criteria for similarity are some dimensionless ratios which depend on the dominant forces acting. Depending on the types of forces to be considered, the scale of the model with respect to all dimensions is suitably chosen. Usually the forces to be considered are (1) *Gravity*, (2) *Viscosity*, (3) *Surface Tension* and (4) *Elasticity*.

The first two factors are usually important in all hydraulic models. It is not possible to satisfy the similarity conditions in respect of all the above factors. Depending on the nature of the experiment, the choice of model dimensions is governed by the dominent forces. The usual dimensionless ratios used in experiments are the Froude number and the Reynold's number, which govern gravity and viscosity forces respectively.

$$\text{Froude number } F=\frac{V^2}{gL}$$

$$\text{Reynold's number } R=\frac{VL\rho}{\mu}$$

where, V=Velocity

L=any length influencing flow

g=accelerations due to gravity

ρ=mass density of fluid

μ=co-efficient of viscosity

It will be seen from the above ratios that Froude's number requires that $V \propto \sqrt{L}$ and Reynold's number $V \propto \frac{1}{L}$. Unless the model scale is 1 : 1 not more than one condition can be satisfied at a time. For models of weirs and dams, where the important force acting is gravity, the Froude number must be the same both for the model and the prototype.

Experiments were conducted with a 1/32 scale model.

The following are some relationships which held good for the model, which was used in all experiments.

Length scale	Lr	=	Lr	=	32
Time scale	Tr	=	$(Lr)^{\frac{1}{2}}$	=	5.7
Area scale	Ar	=	$(Lr)^2$	=	1,024
Velocity scale	Vr	=	$(Lr)^{\frac{1}{2}}$	=	5.7
Discharge scale	Qr	=	$(Lr)^{5/2}$	=	5,792

where r referred to the scale ratio of prototype to model. It will be interesting to note that in the case of weir models, the co-efficient of discharge varies with the size of the model also apart from reasons above mentioned. For clear overfall weirs, the co-efficient is higher, the bigger the model. This is due to sharper stream lines at the top and a consequent steeper pressure gradient. However, in submerged weirs, the behaviour is quite different due to a lesser sharpness in the curvature of the flow over the weir and a flatter pressure gradient— the bigger the model, the smaller the co-efficient. However, by experimenting on two or three scale models it is possible to make a fair estimate of the probable co-efficient in the prototype.

Also, the smallest size of the model to be chosen is limited by the Reynold's number. A Reynold's number of 2,000 denotes the lower critical velocity condition and 2,500 denotes the upper one. The lower value being the true limit, it is necessary that in the model laid in conformity with Froude's law, the Reynold's number should exceed 2,000. So we can establish the following relationship.

$$\text{Re } i.e., \frac{VL}{\gamma} > 2000$$

$$\text{But } \gamma = \frac{0.0000192}{1+0.03368\,t+0\cdot000221\,t^2}$$

where t = temperature in centigrade.

Assuming $t = 20°$

$\gamma = 0.0000075$

So $VL > 2000 \times 0.0000075$ *i.e.*, $VL > 0.015$

FINISHING OF THE MODEL

Froude's law requires that the model ratios should obey

$$V_r = r^{\frac{1}{2}} \quad .. \quad .. \quad .. \quad (6C.43)$$

By Manning's formula,

$$V = \frac{\kappa}{n} . R^{\frac{2}{3}} S^{\frac{1}{2}} \quad .. \quad .. \quad (6C.44)$$

and the prototype model ratios are related by

$$V_r = \frac{R_r^{2/3}}{n_r} \text{ as } \kappa \text{ and } s \text{ are constants} \quad .. \quad (6C.45)$$

(6C.43) and (6C.45)

$$L_r^{\frac{1}{2}} = \frac{R_r^{2/3}}{n_r} \text{ whence in terms of units}$$

$$n_r = (L_r)^{1/6}$$

with the best cement plastered surface we can have n somewhat less than 0·010 and not always as small as that demanded by the law. The model surface was made as smooth as possible, giving it a glossy surface by rubbing with flint stones.

Thus the frictional resistance in scale models is more than that the scale law demands. So the co-efficient of discharge obtained in the model is less than that in the prototype. That is, however, an error on the safe side.

EXPERIMENTAL ERROR

Assuming the usual weir formula, and a unit width $Q = C.L.H^{1.5}$, H including the head due to approach velocity. We may study the behaviour of the co-efficient of discharge with an infinitesimal variation of H. We have, for a given Q, differentiating the equation,

$$H^{3/2}\,\delta C + \frac{3}{2} + CH^{1/2}\delta H = 0 \quad i.e., \quad H\delta C + \frac{3}{2} C\delta H = 0$$

$$i.\,e., \quad \delta c = \frac{3\,Q\delta H}{2\,H}$$

It is seen from the equation that the error in deducing the co-efficient is opposite in sign to that committed in reading the head, *i.e.*, if we overestimate the head we deduce a lesser co-efficient. Also we observe that with the decrease in the head the error in the co-efficient increases. So the error in experiments involving the measurement of smaller heads is more than in the measurement of bigger ones.

A slight error even in the measurement of the velocity of approach will also cause similar errors.

For greater heads, however, the velocity of approach increases and renders the attainment of accuracy difficult.

INGLIS'S EXPERIMENTS

Sir Claude Inglis has done a set of valuable experiments on high co-efficient weirs which has been reported in the annual report of the work done at the Hydro-dynamic Research Station at Kadakvasala for the year 1939-40. It is clearly demonstrated therein that the high co-efficient obtained in weirs is due to the development of negative pressures. The problem is therefore to get the maximum co-efficient for a minimum negative pressure for the required $\frac{D}{H}$ ratio. He has listed a table of high co-efficients for various profile sections for $\frac{D}{H}$ ratios ranging from 0·51 to 1·70. The maximum $\frac{D}{H}$ ratio in the case of the Tunga Anicut is only about 0·35. It is well known that a weir which gives a maximum co-efficient for a particular $\frac{D}{H}$ ratio will give a lesser value for all other $\frac{D}{H}$ ratios. For a $\frac{D}{H}$ ratio of 0·61 he has obtained a co-efficient of 4·83 for a minimum $P/wD\left(\frac{\text{Pressure head}}{\text{Depth of flow}}\right)=2\cdot9$. This means, if this is adopted for the Tunga Anicut, when a 24·4 feet depth of water is flowing over the weir, the co-efficient realised will be 4·83 and the minimum pressure will be $-2\cdot9\times24\cdot4=-70\cdot76$ feet, which is cavitation pressure. As the maximum depth of overflow in the Tunga Anicut is only 14 feet, this section cannot give the expected co-efficient of 4·83. It will be very much less. As this profile neither gives the expected high co-efficient nor keep the pressure within cavitation limit, it is clearly unsuitable.

An examination of other sections proposed by him were also found unsuitable for similar reasons. Hence experiments were necessary to get a suitable profile for the conditions obtaining.

EXPERIMENTS

Experiments were conducted in a three-feet masonry flume the sides of which were well plastered, with effective stilling arrangements. Side contraction was completely suppressed and the side drag effect was minimised by polishing the sides.

$\frac{1}{32}$ scale models were used throughout. Assuming a co-efficient of roughness for prototype $n_p=0\cdot013$, we have

$$n_m=\frac{0\cdot013}{(32)^{1/6}}=0\cdot0073$$

This is smaller than an attainable value. So the co-efficients obtained indicated smaller values than true ones.

In the experiments all discharges were measured on a precalibrated right angled notch. The weir flume was calibrated for velocities of approach for different heads. All the heads were measured with hook gauges reading correct to 0·001 of a foot.

Results were plotted on logarithmic paper because it is the most suitable one for interpreting the results as also for interpolation and extrapolation of results when necessary.

ORIGINALLY PROPOSED PROFILE

It consists of the ordinary trapezoidal profile with a height of 40 feet, a top width of 10 feet with front and rear slopes of 12 in 1 and 2 in 1 respectively. Experiments on a 1/32 scale model showed that the maximum co-efficient was 3·43 for a $\frac{D}{H}$ ratio (D, the head and H, the height of the weir) of 0·55, *i.e.*, for a head of 22 feet in the prototype. The co-efficient for 13·75 feet head (which is the maximum head and assumed in the prototype) is 3·105.

HIGH CO-EFFICIENT WEIR PROFILE

Experiments were conducted on circular, elliptical and parabolic top curves of the same parameter and the same rear glacis. The parameter was, however, arbitrarily chosen, consistent with the stability of the structure. The front was kept vertical and the rear glacis was $1\frac{1}{2}$ in 1 slope. The results obtained were as follows :

Shape	$\frac{D}{H}$ *Ratio for Max. C.*	*Max. C*
Circular	0·52	3·89
Elliptical	0·64	3·93
Parabolic	0·56	4·26

Thus it is seen parabolic top gives a higher co-efficient than circular and elliptical ones.

REAR SLOPE

Next it was proposed to find the effect of the slope of the glacis on the co-efficient. The same top curve was adopted throughout and the join of the curve and the glacis was rounded off. The maximum co-efficient is obtained for a slope of $1\frac{1}{4}$ in 1. The corresponding D/H ratio, however, varies from slope to slope according to no regular law, but for any particular D/H ratio the rear slope for maximum co-efficient may be selected.

The following remarks may be of interest. A glance at Figure 6 C.28 shows that the nearest straight line to the Creager's curve of sufficient height will have a slope $1\frac{1}{2}$ in 1 ($1\frac{1}{2}$ horizontal and 1 vertical), and from pressure conditions, apart from others, the same slope may be advantageously adopted for dams of sufficient height.

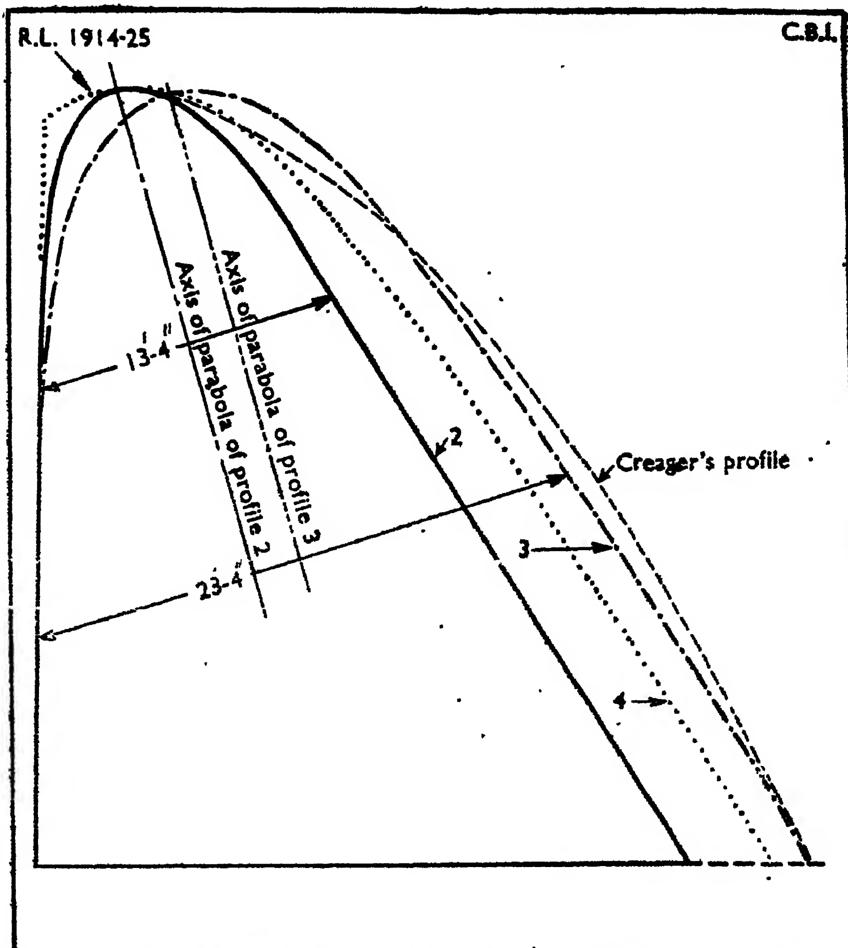

Figure 6 C.28

CONSTRUCTION OF THE TOP PARABOLA

Construct an angle AOC so that tan AOC$=1/1\frac{1}{2}=2/3$. Bisect the angle AOC and draw the bisector OB. Draw a line perpendicular to OB such that the offset to one limb$=x$. Let it cut the bisector at B.

Bisect OB at P. Then P is the vertex of the parabola.

Then draw the parabola APC, (Figure 6 C.29) which is tangential to both the limbs.

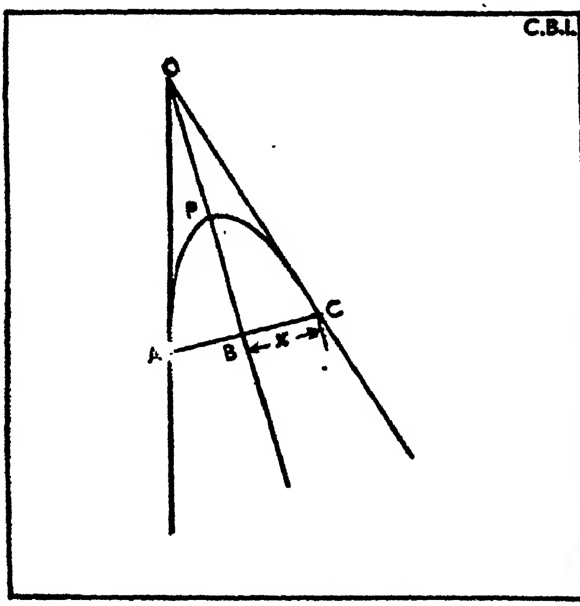

Figure 6 C.29

PARAMETER.

It was proposed next to investigate its behaviour with different parameters of the parabola the rear slope being kept constant ($1\frac{1}{2}:1$). Four different parameters were tried on a $\frac{1}{32}$ scale model. It was seen that the peak point was maximum for the parabola $x^2=1 \cdot 514\, y$, the vertex being the origin and the axis being the X-axis. For smaller parameters, the nappe tries to separate and the maximum co-efficient obtained is limited. In higher parameters, the friction at top will be more, the curvature of streamlines at top will be less and hence the maximum co-efficient attainable is limited.

PRESSURE.

As previously mentioned, no design of a high co-efficient weir is complete without testifying to the fact that for the maximum obtainable D/H ratio and discharge conditions the pressure does not reach cavitation limit.

PROTRUDING LIPS ON TOP OF WEIRS.

Very often it is seen in high co-efficient weirs that a protruding lip is designed at the top. Though such an arrangement alone cannot improve the co-efficient of discharges in any appreciable proportion a lip helps to secure the following advantages :—

(1) Besides keeping the designed high co-efficient weir profile at the top, masonry can be saved in front of the weir.

(2) There is a negligibly slight increase in the co-efficient of discharge owing to the elemination of currents from bottom and that only when the depth of the lip (the thickness) is more than the depth of spillage.

SCOUR PREVENTERS

There are many remedies for the prevention of scour below weirs. The following important ones may be mentioned :—

(1) Aprons.

(2) Baffles—Rehbock dentated sill is a special type.

(3) Water cushion.

(4) Creation of the hydraulic jump.

(5) Formation of Hyhraulic Rollers.

HYDRAULIC ROLLERS

The most natural and perhaps the most economical remedy is to induce such hydraulic rollers as help pile detritus at the toe of the dam and push the formations of the pool downstream. Butcher and Atkinson have made an extensive study of hydraulic rollers and their theoretical investigation on minute models has been of a great help for us in the design of dam profiles in order to induce desirable hydraulic rollers.

There are two kinds of vortices, positive and negative. When water flows from left to right, clockwise vortices are termed positive and anti-clockwise ones negative. Both kinds of vortices may occur simultaneously at the toe of the weir. For a particular weir, the nature of the vortices are defined and they determine the scour conditions. Also the lines of flow are analogous to magnetic lines of force and at any point both the direction and the velocity are defined. There will be neutral points such as P which are subjected to neither piling nor scouring. In a rectangular channel, the locus of the neutral surface such as AB which is usually of the form $\frac{y}{V^2/2g} = A\left(\frac{x}{V^2/2g}\right)^B$.

That is, the stable surface is subjected to neither accretion nor scouring. That shows the nature of the bed that would form were the bed loose and easily carried by water.

Several prototype investigations bear testimony to the fact that the model observations are qualitatively quite representative though often it is not quite possible to simulate conditions quantitatively in models.

UPTURNED BUCKETS

To simulate the required hydraulic rollers suitably designed upturned buckets are very useful. These buckets not only add to the stability of weirs but also help to relieve the bed from the impact of water.

DESIGN OF UPTURNED BUCKETS

According to Schoklitsch, the minimum bucket radius for deflecting the water jet reliably is equal to 0·15 times the height of the weir. Extensive experiments of local nature have been conducted at the Poondi Research Station with regard to the Ramapadasagar overflow dam. The design of the upturned bucket depends on the downstream levels for different discharge conditions. Accordingly, however, there are two kinds of buckets. One is the shooting bucket with a correspondingly great radius of curvature and a lesser angle of deviation. The other is the deep bucket with a small radius of curvature and a greater angle of deviation.

The design depends on the particular problems to be dealt with and the velocities and depths obtaining therein.

UPTURNED BUCKET FOR THE TUNGA ANICUT

For lack of extensive gauge records, a final decision about the upturned bucket has not been possible. However, assuming the maximum discharge conditions as upstream level 1,930.00 and downstream level 1,902·00 a design has been arrived at for favourable results. The bucket will have a radius of 10 feet its invert 4½ feet above bed level the shoot of the water being at 45° to afford maximum range.

Extensive experiments have been carried out to that effect, in a 3 feet wide glass flume, scour patterns being obtained for varied conditions of flow on a graded sand bed in rear of the anicut model.

Levels are gauged by means of moving trolley with a point gauge fixed on it. Upstream and downstream levels are regulated and the model is run for one hour—a length of time sufficient for qualitative reproduction of the prototype.

Later, the sand bed is drained out and the contours gauged by means of the trolley and gauge.

It may be mentioned that the stable surface is independent of the scale chosen unless the material is actually immovable. Bucher and Atckinson have experimented with sand and gravel and they have testified to the fact. Again, they have made studies on the prototype structures also and come to the same conclusion.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. S. N. GUPTA introduced items (1) to (3). He said that at the 17th Annual Meeting of the Research Committee a reference was made to the design of a battery of volute siphons discharging into a common tunnel. The design had since been worked out. The battery consisted of four volute siphons of 12

feet diameter each with a gradually expanding common elliptical tunnel with an outlet area four times the individual barrel area. The working head was 26 feet. The outlet ran full bore; the co-efficient of discharge was observed to be 0·7 and the priming depth one foot.

Basic work had been taken up to standardise the design. Some of the interesting conclusions were :—

(1) The optimum spacing between the volutes was 4D for maximum co-efficient of discharge. The priming depth was minimum for a comparatively small spacing.

(2) The rectangular outlet tunnel gave the minimum priming depth as was expected.

(3) The optimum number of volutes was found to be four.

Regarding item (2) MR. GUPTA said that a 15 feet by 10 feet saddle siphon with a volute priming siphon for Lalitpur Dam was designed and tested in a model. This design was similar to the one previously designed for Pahuj weir with a few minor alterations. It was confirmed once again that to ensure better priming qualities in a saddle siphon of big cross-sectional area, provision of a lip was essential in addition to a volute priming shipon.

In introducing item (3) MR. GUPTA said that the basic study on the volute siphon was continued during the year, maximum co-efficient of discharge and minimum priming depth were indicated when the diameter of the covering hood was 3·33 times the barrel diameter.

RAO BAHADUR D. V. JOGLEKAR introduced items (4) and (5). As regards item (4) he said that a half-size model of the gate and the waste weir were constructed in the eight feet wide flume, and experiments were carried out on behalf of Messrs. Duncan Stratton & Co., Bombay. He then explained the details of the experiments and results obtained.

In reply to a question by the Secretary, RAO BAHADUR JOGLEKAR said that it was at the highest flood level of T.H.D. 422·25 of the reservoir that the gate started falling. Upto 422·00 nothing happened.

Regarding item (5) RAO BAHADUR JOGLEKAR said that experiments had been carried out previously to dissipate the energy of the jets by making jets from adjacent siphons hit each other—after they left, the outlet—at various angles. This method was, however, found unsatisfactory.

In the experiments carried out this year, jets from individual siphons were dispersed by providing suitable short length expansion pieces to the outlets. The expanding portion or disperser had a central wedge shaped ridge gradually increasing in height away from the outlet. The height of this ridge at the end

of the disperser was about 1/4th the diameter of the siphon outlet. The disperser effected a fanning out of the jet, thus reducing the intensity of impact with the water downstream. To further fan out the jet a tilting ledge was added at the end of the disperser ; the angle of the ledge could be varied to obtain opitmum effect ; various dispersers in combination with ledges at different angles were tested.

In reply to the Secretary RAO BAHADUR JOGLEKAR said that the reduction in co-efficient of discharge was from 0·77 to 0·75. The maximum depth of scour in the original was 0·526 foot. It was reduced to 0·106 foot by this method.

MR. T. P. KUTTIAMMU introduced items (6) to (13) and said that he did not have anything special to add except on the stilling basin evolved for the Lower Bhavani Dam.

MR. KUTTIAMMU referred to Fig. 59 (*a*) on page 77 of his report and said that the design was a simple stilling basin with the addition of a single row of floor blocks inside the basin. This was found to be extremely efficient. The floor blocks could be buttressed against the end wall for giving structural stability and strength.

As Mr. Hardiker was not present item (14) was taken as introduced.

DR. J. K. MALHOTRA introduced item (15) and said that in the design of siphon spillways the jet should strike the river bed as far from the work as possible. This would ensure that bed scour close to the work was avoided. It was, therefore, sometimes necessary to incline the nozzle upwards. He referred to Naylor's formulas for optimum inclination θ of the nozzel, which would throw the jet to the maximum distance l from the work, for a given exit velocity l and a nozzel height a.:—

$$\text{Cosec } \theta = \sqrt{\frac{2ga}{v^2} + 2}$$

$$\text{And } l = \frac{v^2}{2g} \text{Cos } \theta \left(\text{Sin } \theta + \sqrt{\text{Sin}^2 \theta + \frac{2ga}{v^2}} \right)$$

which DR. MALHOTRA said that these formulas were rather cumbersome and he had considerably simplified them. He also demonstrated how they could be reduced to a very simple geometrical construction, which gave an exact solution for land.

MR. N. S. GOVINDA RAO introduced items (16) to (20) and said that experiments on dissipating the energy of flow from siphon outlets were continued. Three methods had been tried last year, one by impinging jets, another by expanding the flow from circular to an elliptical one, and by providing helical guide vanes at the outlet. Three more methods were tried this

year. One was by supporting the jet on an ogee curve. This was expansive and the flow did not fan out well. Next a standing wave device was tried. This was satisfactory but costly. The third one was to keep the outlet practically submerged at the tail water level before priming, leaving only a small gap for the air to escape. This has proved very effective. In the eighteen inches model tested the priming depth was only ½ inch as against the usual three inches.

Introducing item (17) Mr. GOVINDA RAO explained how the design of volutes were arrived at. Regarding the design of volutes two things were important—one was that the filaments of flow from the lip should flow along the funnel and in a point forming a "boil" which should be within the vertical barrel itself. The other point was that the boil so formed should have a special motion so as to screw the air. The design is such that the air is trapped in four ways :—(*a*) by boundary air effect, (*b*) by screwing action of the water seal, (*c*) by physical entrapoment of air between planes of flow in the successive gaps between two volutes and by (*d*) formation of spray due to impinging of fins created by the water filaments striking the nose of water.

Model experiments had been carried out to find the optimum height of the volutes and to find priming depths.

Mr. GOVINDA RAO said that a knowledge of aerodynamics was necessary for the solution of the mathematics flow through siphons.

Regarding item (18) Mr. GOVINDA RAO said that there was a school of thought that the discharge in a siphon was limited to a difference of 34th of water levels upstream and downstream of a reservoir. What was actually limited was velocity with which water could enter a siphon. Allowing for friction and other losses the net effective atmospheric head may be taken as 25 feet which corresponds to a velocity of 40 feet per second. This is the maximum velocity with which water can be pushed in by the atmosphere into a siphon inlet. If you give sufficient area at the inlet such that the total discharge at the inlet is equal to the discharge capacity at the outlet. There is no reason why a siphon should not flow full for all heads. The design is in the areas you give at the inlet and outlet. Experiments made at the 60 feet head siphon at Shimsha had been sucessful to certain extent. The priming depth for a 10 inch diameter siphon was less than ½ inch.

MR. V. N. NAGARAJA introduced item (20) and said that the problem was to design a suitable profile for the proposed Tunga Anicut at Sakrebyle in Mysore State. In order to discharge about 250,000 cusecs over a 40 feet high and 1,700 feet long weir with a spillage depth of 14 feet. The Khadakvasla Research Station had given a number of high co-efficient weir profiles in their Annual Report for 1939-40. None of them could be adopted because they not

only did not give the expected high co-efficient for the $\frac{D}{H}$ obtaining ratio there, but also the negative pressures developed were high. Though high coefficient and negative pressures went together, and attempt to correlate them from the readings recorded in the Khadakvasla report and from readings taken at Krishnarajasagar was not successful. The reason might be that the maximum negative pressure recorded were not really the maximum ones but only approximately so.

It was necessary to design a new profile. The Creager's profile was not recommended because it was desirable to attain a higher co-efficient, allowing a reasonable maximum negative pressure if necessary.

An entirely new approach was made towards the problem. While it had been usual to determine the co-efficients on different curves of different parameters chosen at random that did not help much in the application of the result for a new set of conditions. A general approach was made at Krishnarajasagar. First of all the circular, the elliptical and parabolic curves of the same parameter were compared. The parabola having given the best result parabolas of different parameters were compared. All other factors remaining constant, the behaviour of the co-efficient of discharge with respect to the rear slope was studied. Proceeding on these lines, a design was recommended for the Tunga Anicut.

The secret of the high coefficient weir lay in the development of negative pressures at the crest *i.e.*, in making the filaments of water take sharp curvatures at the crest. How far below the rest the surface tension should be developed between the water filament and the masonry profile was a very interesting point for study. In the experiments, allowance was made for the velocity of approach.

MR. S. N. GUPTA said that it was high time to decide whether the volute siphons could be used for high heads. In the Hirebhaskar the head was 60 feet and the siphon did not work during the last monsoon. Marconahalli siphon had been reported to run full bore, but the prototype co-efficient of discharge was never determined. Mr. Gupta was of the opinion that these volute siphons could not operate for very high heads ; the maximum limit might be as high as 50 feet in consistent with both economy and efficiency. The theory put forward by Mr. Govinda Rao that once the funnel and the barrel was filled with water, the working of the siphon was automatic, does not appear sound at this stage.

RAO BAHADUR D. V. JOGLEKAR referred to item (18) and said that the results obtained were very interesting. The low co-efficient obtained in the high head siphon at Shimsha was apparently due to insuction of air on account of the low capacity of the inlet chamber. To get reliable results it was necessary to have a chamber having at least 10 to 15 minutes' supply. Such

experiments were really very costly. It was, however, now proposed to construct two experimental models in the Hirakud Dam where the discharging capacity, pressure, cavitation and its effect on other factors etc., would be very carefully observed.

MR. V. N. NAGARAJA said that for any experiment to be decisive it was absolutely necessary that the experimental siphon should be more than 70 feet high so that the effective operating head should come to more than 30 feet.

MR. N. S. GOVINDA RAO said that regarding Mr. Gupta's point everything had to be experimented upon at least once. A start was made by constructing a siphon of two feet diameter, which figure had now been increased considerably. Research was still being carried out on that point and the results were not yet final. A whole book had been written on the subject. One of the experiments cost 15,000 rupees. These experiments were no doubt very costly. The point was to convince everybody. The main consideration was more about the priming depth than about the co-efficient of discharge.

THE SECRETARY asked Mr. Govinda Rao to intimate the difficulties met with in the Hirebhasgar siphon last year.

MR. GOVINDA RAO replied that at Marconahalli a siphon of eight feet diameter operating under a head of 40 feet primed at only eight inches. He enquired of Mr. S. N. Gupta whether he had any data on which he based his opinion.

Some experiments were conducted with models of siphons proposed at Hirebhasgar. It was concluded from the model experiments that the priming depth would be twelve inches. The trouble was that in the model, any small error made did not seriously vitiate results. The volutes in the prototype right up to the throat of the funnel were one foot thick. When water began to flow between the volutes there was a gap corresponding to the thickness of the volutes between the flow on either side of it, through which air rush in and break the formation of a water seal. The siphon did not prime till this was corrected.

RAO BAHADUR D. V. JOGLEKAR said that the Central Waterways. Irrigation and Navigation Research Station had already recommended that the section of the outlet should be contracted from 18 feet to 16 feet so as to dissipate the head in excess of the effective atmospheric pressure. It was learnt that when attempts on similar lines were made at Hirebhasgar, then only these siphons were able to function.

THE CHAIRMAN (RAI BAHADUR S. D. KHANGAR) asked how the silt was excluded through the siphons. Does silt not get collected at the bottom of the reservoir ?

MR. N. S. GOVINDA RAO replied that siphon was an excellent instrument for sucking out silt. For removing silt there was nothing like a siphon.

THE SECRETARY explained some of the energy dissipating devices adopted in France and mentioned particularly the design adopted at the L. Aigle Dam.

MR. N. S. GOVINDA RAO referred to item (5) and said that the ingeneous device of adding a disperser appendage seemed to have given hopeful results for dissipating energy. Instead of a straight divergence for the disperser a hyperbolic divergence might give a smoother flow and therefore a greater co-efficient of dishcharge.

Since the flow came out with a twist it was quite probable the flow between the two halves of the disperser might not be evenly distributed in the prototype. This rotary motion of the flow might set up turbulence at the mouth of the outlet locking up air. This effect which was insignificant in a model becomes quite pronounced in the prototype, *e.g.*, a ledge was given in a siphon at Hirebhasgar last year which resulted in a quantity of air getting locked up in the siphon, although this was not noticed in the model. The observation regarding the effects of this on the priming depth would be useful data.

Experiments had been conducted with a considerable depth of water cushion on the downstream side. In a heavy water cushion it was quite possible that positive vortices were set up which tended to pile up detritus at the toe of the body wall. So it was very important that experiments were conducted for low downstream water levels and a study made whether there would be retrogression of levels by scour, and if so whether the retrogression would extend to the toe of the body wall.

The addition of a ledge to a disperser might make the whole appurtenance very costly. All things considered, probably it would be cheaper to keep the outlet of the siphon almost at the bed level of the river so that it got submerged under the tail water after its priming. The reduction in the priming depth and the savings effected in protective work would more than offset the disadvantage due to reduction in the co-efficient of discharge.

Regarding item (15) MR. GOVINDA RAO said that an attempt had been made in his paper to derive a dimensionl ess equation for maximum length of trajectory of flow. In the prototype Froude's number was the dominant factor. As the discharge in a siphon had a considerable quantity of air in it, Reynold's number was not without significance. If it exceeded a million, as it did in siphons working under high operative heads, the turbulance was so great that water began to give up the air in it during its descent in the vertical barrel and these might cause formation of air pockets. This limitation of Reynold's number not exceeding a million should not be lost sight of in designing the prototype.

In model experiments, care is always taken to have the size of the model big enough to give a Reynold's number in excess of 2,000. It is, however, seen that in very small models Weber's number is also significant. A mere sharpening of the edge at the throat of the funnel reduced the priming depth. If the edge at the funnel edge was not sharp, water hugged the surface of the funnel and barrel all round without meeting to form a water seal.

As regards item (1) Mr. Govinda Rao said that the experiments conducted the same area of outlet was used for experimenting on two, three or all the siphons. When less than four siphons were operated the conditions prevailing were those of diverging outlets and consequently a higher co-efficient of discharge was realised in the model. It might be recalled that in the prototype the efficiency of siphon with expanding outlet was much less than in the prototype

It had been deduced *vide* H.R.S.K. report 1947, pag 31, that the operating head should be more than about 3·1 times the barrel diameter for good priming. The present design, where the operating head was 26 feet and the barrel diameter 12 feet, falls too short of the required proportions to effect a perfect water seal.

In a design of this type where several siphons discharge into the same tunnel, the co-efficient of discharge in the model was lower and priming higher than the usual type. Pressure observation would be very useful.

A single elliptical siphon did not prime under normal conditions, for the priming depended on the vortex flow inside the funnel and barrel and its capacity to evacuate air from the dome. A vortex being always circular, an elliptical barrel was definitely an impediment and also the shape was not conducive to the formation of a perfect boil, for the trajectories of water from the bottom of the funnel had to travel different horizontal and vertical distances before they met.

As to item (2), Mr. Govinda Rao said that the introduction of the lip to improve priming qualities was dangerous because it reduced the efficiency of the siphon, which was not very well indicated in the model. A better remedy would be to have a bigger priming siphon. As a rule, in saddle siphons of uniform cross-sectional area and of heads about 20 feet, a smooth curve was necessary at the crest. Otherwise, the crest became the bottleneck, in this case the cross-section at the lip limited the discharge because of the sharp radius.

Referring to item (3) Mr. Govinda Rao went on to say that the data collected were no doubt very useful. The adoption of the optimum dimensions given for the hood diameter lessened the discharge per unit width of spillway. The optimum result of 3·3 arrived at might not hold good for all operating heads.

The graph No. 16 had been prepared for a particular operating head and the same optimum ratio of Hood diameter/Barrel diameter had been used for other operating heads to study the behaviour of the priming depth with varying

operating heads. It was worth-while studying whether the same optimum limit of Hood diameter/Barrel diameter=3·33 gave optimum priming depth and co efficient discharge for other heads also.

For the optimum spacing of domes, velocity potential lines had to be drawn and the variation of strength up to the neutral points found out. Mr. Govinda Rao felt that in the design of siphons guidance had to be sought from experiments done in aerodynamics on the subject of flow from sources into sinks and the laws governing the formation of vortices *etc*.

Mr. S. N. Gupta replying said that Mr. Govinda Rao seemed to have misunderstood the divergence portion of the siphon. Drawing No. 15 of the United Provinces report showed that area of the outlet was exactly equal to the siphon area when one, two or three siphons were added to the battery. The properties of the diverging outlet did not come in the picture at all in this case. As to the working of the battery, Messrs Khosla, Gulhati and Bose had seen these experiments and Mr. Gupta thought that they were all satisfied. They were thoroughly convinced that lip was an essential part of the siphon for ensuring early priming and better performance on the whole. After all there was a limit to the size of the priming siphon.

Dr. J. K. Malhotra said that he was grateful to Mr. Govinda Rao for pointing out that the flow influence could also be taken into consideration.

Mr. N. S. Govinda Rao said that Dr. Malhotra's equations were limited by the value of the Reynold's number.

It was decided to keep the subject on the Agenda.

DISCUSSION BY THE BOARD

The Secretary said that 20 items were discussed at the Research Committee Meeting. There was no resolution on the subject.

Rai Bahadur C. L. Handa pointed out that drum gates which were required at spillways could be manufactured in India. At the Central workshops, Amritsar the detailed designs that were obtained from the International Engineering Company had been examined in detail and the opinion was that they could be manufactured in India. This step would save a lot of trouble and would conserve our resources as far as possible. Questioned as to what length of the spillway the drum gates could be made Rai Bahadur C. L. Handa said that for about 100 feet long it may be possible—same as for Grand Coulee Dam.

Dr. H. L. Uppal mentioned something about the design of the buckets. They had used a radius of 50 feet. Experiments were carried out at Madhopur Station both on sloping and horizontal aprons. It was found that the horizontal apron downstream of length 10 to 12 feet in length was good enough. In the design of falls also horizontal aprons of length=4 times depth of water downstream worked well. The experiences of the Bureau of Reclamation also showed that horizontal aprons are preferable. Even in the case of shooting bucket they were satisfactory.

7C. Hydraulic Structure on Permeable Foundations

PRELIMINARY NOTE

This subject was on the agenda of the Research Committee from 1930 till the time of the issue of Board Publication No. 12, 'Design of weirs on permeable foundations' in 1936.

The design of pavements of regulators on clay and sand clay foundations, was also on the agenda of the Research Committee. This subject was removed from the agenda in 1941 with the following resolution passed by the Board.

(a) " Resolved that this subject be taken off the agenda, but in order to complete the information published by the Board, the two notes by Rai Bahadur Khosla be published as a supplement to Board Publication No. 12 (with any modifications which the author may consider necessary).

(b) " The Board is of the opinion that basic research on flow under dams where the sub-soil is not homogeneous, is not likely to be useful in view of the diversity of formation which may possibly occur and the difficulty of reproducing those conditions in models.

(c) ' In cases where effectively impervious strata of soil occur, *ad hoc* experiments might be carried out to determine the effect on sub-soil flow in each particular case.

(d) " In view of this, the subject 'The design of pavements of regulator on clay and sand clay foundations ' should be removed from the agenda. "

No action was taken on part (a) of this resolution. C. B. I. Publication No. 12, is now to be revised and it has been decided that the developments that have been achieved since its publication should be included in the revised edition. The Board decided in 1946 to re-introduce the subject on the agenda of the Research Committee from 1947. The following items were discussed at the 1947 Research Committee Meeting :

(1) Pressures on Lloyd Barrage floor

(2) The Poondi regulators.

Recent Literature.

(1) Bezdick, Vilibald., Ingenieur, Docteur, Counseiller de section au Ministere technique, Prague.—Les Sous-Pressions Dans un Barrage en terre

Barrage de Frystak (Uplift on an earth dam. Large dam at Frystak).—International Commission on Large Dams, Third Congress Stockholm, 1948, R 5.

(2) Kratochvili, Stanislav, Ingenieur, docteur, professeur. Haute Ecole Technique Bratislava.—Sons Pression sur les fondations du Barrage de Kninicky (Uplift acting on the base of the dam at Kninicky).—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 6.

(3) Myslivec, Alois, Docteur, Professeur a l' Institut d' Hydraulique Prague-Podbaba Tchecosl.—Infiltrations a travers les digues on terre employees pour les barrages reservoirs et terres propres a leur etanchement (Seepage through earth-dams of water reservoirs and soils suitable for the waterlight layer).—International Commission on Large Dams, Third Congress, 1948, R 7.

(4) Speedie, Milton G., M.C.E., A.M.I.E. Aust., Assoc. M. Am. Soc. C.E.—Experience gained in the measurement of pore pressures in a dam and its foundation.—International Commission on Large Dams, Third Congress, Stockholm, 1948, R12.

(5) Leliavsky Bey, Serge, Ph.D., M.Inst. C.E., M.A.S.C.E., Director, Designing Service, Reservoirs and Nile Barrages, Ministry of Public Works, Cairo.—Pore versus crack as basis of uplift concept.—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 13.

(6) Laporte G., Ancien eleve de l' Ecole Polytechnique Ingenieur-Conseil.—Controle de l' etancheite des fondotions d' un barrage (Control of the imperviousness of the foundation of a dam).—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 29.

(7) Mc-Henry Douglas, Head, Structural Research Section, Bureau of Reclamation, Denver, Colorado, U.S.A.—The effect of uplift pressure on the shearing strength of concrete.—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 48.

(8) Jakobsen, B.F., Special Assistant on Design and Construction to Division Engineer, South Pacific Division, Corps of Engineers, U.S. Army.—Critical exposition of the measurements of uplift pressures and stresses arising therefrom.—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 52.

(9) Harza, L.F.—Uplift Area in dams.—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 53.

(10) Reinius, Erling, Consulting Civil Engineer, Vattenbyggnadsbyran Stockholm.—Effect of Hydrostatic uplift on stresses in concrete and on the stability of dams.—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 57.

(11) Hill, H. Prescott, M. Inst. C.E., Great Britain.—Erosion within or beneath water-retaining structures.—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 1.

(12) Zaki Bey, Hassan, Ph.D., M.Inst. C.E., General Inspector of Irrigation, Upper Egypt and Leliavsky Bey, Serge, Ph.D., M.Inst. C.E., M.A.S.C.E. Director, Designing Service, Reservoirs and Nile Barrages.—Tail-erosion as a factor affecting the safety coefficient against piping.—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 4.

(13) Pospisil, Joseph, Ingenieur, Conseiller superieur de section au Ministere technique Prague.—Dispositions les plus recentes pour empecher la formation des renards (Newest methods for the prevention of defects caused by the penetration of water under the foundations of a dam).—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 9.

(14) Bazant, Zdenek, Ingenieur, Docteur, Professeur a la Haute Ecole Technique, Prague, Tchecosl.—Critical head for piping beneath weirs.—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 10.

(15) Werner, P. Wilh, and Ljung, Egil, Vattenbyggnadsbyran (VBB) Consulting Engineers, Stockholm.—Method of preventing piping at Trayd Power Plant.—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 18.

(16) Mayer A., Inspecteur General des Mines.—Dispositions les plus recentes pour empecher la formation des renards (The most recent precaution to avoid the formation of pipings).—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 22.

(17) Delattre, Ingenieur en Chefdes Ponts et Chaussees, Directeur Technique de la Compagnie Nationale du Rhone,—Dispositions pour empecher la formation de renards (Prevention of pipings).—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 23.

(18) Coyne A., Fondation d' un barrage sur l' alluvion le barrage du Laurenti (Alluvial foundation of Laurenti dam).—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 35.

(19) Ischy, E., Ingenieur E.P.Z. Directeur general de l' Entreprise de Fondations et Travaux Hydrauliques France.—Barrage de Castillon Lutte Contre les erosions souterraines (Castillon Dam—fighting underground erosions).—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 36.

(20) Ischy, E., Ingenieur E.P.Z. Directeur general de l' Entreprise de Fondations et Travaux Hydrauliques, France.—Digue du lace Noir Lutte contre les erosions souternaines (Lac Noir Dam).—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 37.

(21) Rice, O.L., Engineer, United States Bureau of Reclamation, Denver, Colorado and Arthur, H.G. Engineer, United States Bureau of Reclamation, Denver, Colorado.—The most recent methods developed to avoid piping or blow-outs in Dams.—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 49.

(22) Lewin, Joseph D., Civil Engineer, Headquarters Department Board of Water Supply, City of New York.—Prevention of seepage and piping under dams built on permafrost and related problems connected with the design and construction of such dams.—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 66.

(23) Lowe-Brown, W.L., D. Eng., M.I.C.E.,—British Practice in Dam Foundations.—International Commission on Large Dams, Third Congress, Stockholm, 1948, C 2.

(24) Burwell, Edward B., Jr., Chief, Geology and Geophysics Branch Office, Chief of Engineers, U.S. Department of the Army, Washington, D.C.—Foundation Engineering for Large Concrete Dams.—International Commission on Large Dams, Third Congress, Stockholm, 1948, C 6.

(25) Bazant Jr. Zdenek—Critical head for the expansion of sand on the downstream side of weirs.—International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, 1948, Vol. II.

(26) Huizinga, T.K.—Two failures with cut-off walls.—International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, 1948, Vol. II.

(27) Barron, Reginald, A.—The effect of a slightly pervious top blanket on the performance of relief wells.—International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, 1948, Vol. IV.

THE YEAR'S WORK

The following items were discussed at the 1948 Meeting of the Research Committee :—

(1) Model Investigations for the proposed Mor Barrage—Sectional Model.

(2) **Lines of flow under causeways on sand foundations.**

(1) MODEL INVESTIGATIONS FOR THE PROPOSED MOR BARRAGE—SECTIONAL MODEL [1]

ABSTRACT

The design of the Mor Barrage, was tested in sectional models for—

(1) its stability against uplift pressure,
(2) determination of necessary protection like friction bolcks to be constructed on the downstream floor.

The effect of these friction bolcks will be to reduce the downstream scour within safe limits and to increase the water-cushion on the downstream floor. Sections of both the undersluice section and the weir portion were to be tested in models. Gives results of experiments.

THE MODEL (WEIR SECTION)

The design of the weir portion is shown in Figure 7 C.1.

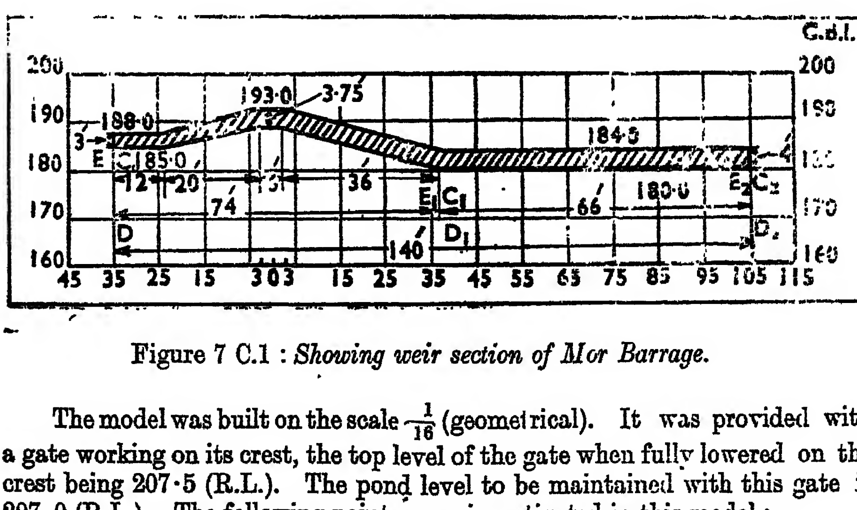

Figure 7 C.1 : *Showing weir section of Mor Barrage.*

The model was built on the scale $\frac{1}{16}$ (geometrical). It was provided with a gate working on its crest, the top level of the gate when fully lowered on the crest being 207·5 (R.L.). The pond level to be maintained with this gate is 207·0 (R.L.). The following points were investigated in this model :—

(1) Testing the proposed design for uplift pressure and surface flow.
(2) Determining the shape and design of friction blocks on the downstream floor for reduction of scour downstream to a safe limit.

DATA

The barrage was designed for an afflux at three feet at high flood level which was given to be 209 feet (R.L.) (downstream of the barrage) for a discharge of 228.000 cusecs in the river.

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[1] River Research Institute, West Bengal, Annual Report, 1947, pages 86—96.

Water level at the proposed barrage site at Tilpara ghat was never observed except during the flood season of 1946. No relia ble data were, therefore, available giving the water levels at the barrage site, for different discharges. Approximate figures were taken as given in Table 7 C.1.

TABLE 7 C.1

River discharge (cusecs)	Expected water (R.L.) downstream of the Barrage		Corresponding discharge per foot run over the weir section in cusecs per foot run
	Before retrogression	After retrogression of 4 feet	
125,000	205·0	201·0 ..	100
228,000	209·0	205·0 ..	200
300,000	212·0	208·0 ..	300

EXPERIMENTS

The experiments were conducted on the following lines :—

(1) Upstream water levels were determined for different discharges passing over the weir under free-fall conditions. The discharge co-efficient of the weir section was calculated from these experiments.

(2) A discharge equivalent to 300 cusecs per foot run was run over the model with the gate fully raised and water profile over the structure was measured maintaining different downstream water levels ranging from 208·0 to 198·0 (R.L.). The model was run for eight hours in each of these cases and the shape of the scour downstream of the *pucca* floor was measured.

(3) Similar experiments to those in (2) above were carried out with discharge equivalent to 200 cusecs per foot run, the downstream water level being maintained at 202·0, 200·0 and 198·0. Water profile only was taken also for downstream water levels at 206·0 feet and 204·0 feet.

(4) Similar experiments to those in (2) above were carried out with discharge equivalent to 100 cusecs per foot run, the upstream water level being

maintained at 207·0 (R.L.) by application of the gate for different downstream water levels ranging from 203·0 to 193·0 (R.L.). Scour was not measured in these cases.

(5) The set of experiments mentioned in (2), (3) and (4) above were run on the model as designed and with different sets of friction blocks fixed on th downstream floor. Two rows of staggered blocks (rectangular shape) wer fixed at the toe of the glacis and two rows at the end of the *pucca* downstream floor. Three different heights of these blocks (3 feet, 3·5 feet and 4 feet) were tried. The arrangement of blocks on the downstream floor is shown in Figure 7 C.2.

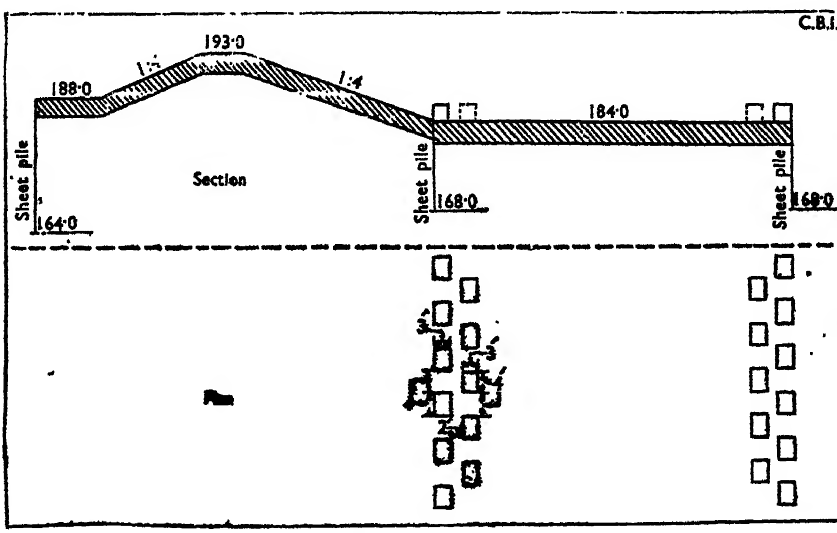

Figure 7 C.2:—*Showing friction blocks in position.*

ANALYSIS OF EXPERIMENTAL RESULTS

(1) *Discharge co-efficient.*—When this section is used as a weir without gates, the discharge co-efficient (under free-fall condition) for different discharges as obtained from the model (scale $\frac{1}{16}$) is shown in Table 7 C.2.

(2) *Effect of friction blocks on scour.*—The following three systems of friction blocks (staggered) were tried :—

(*a*) Two rows at the toe of the glacis and two rows at the end of the *pucca* floor, size of each block being 4 feet (length) ×3 feet (breadth) ×3 feet (height).

(b) Same as (a) above with blocks of size 4 feet × 6 feet × 3·5 feet (height).
(c) Same as (a) above with blocks of size 4 feet × 3 feet × 4·0 feet (height).

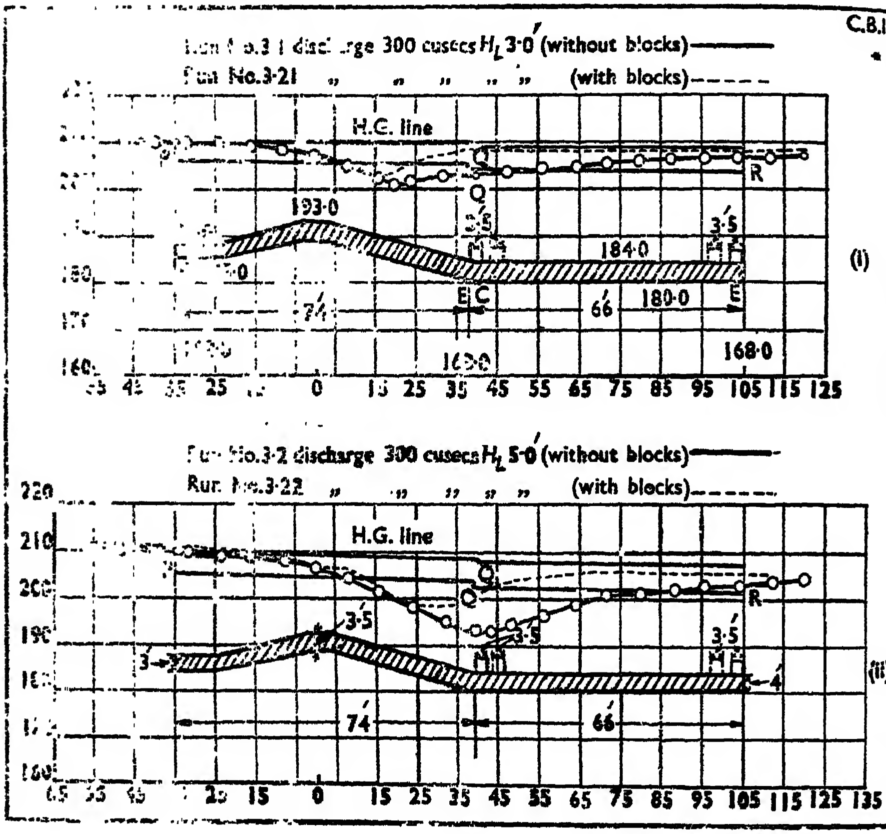

Figure 7 C.3 (i) (ii): *Weir section of the Mor Barrage showing the water profiles observed on the model with and without blocks.*

Scour could not be appreciably reduced by increasing the height of the blocks from 3·5 to 4 feet. The expected downstream water level for this river after a retrogression of 4 feet is 202·0 feet (R.L.) for the normal maximum discharge (150,000 cusecs). The corresponding depth of water above the downstream floor is (202—184), that is, 18 feet; the height of the blocks ordinarily should not be more than 3·5 feet unless appreciable reduction of scour is effected thereby.

(3) *Effect of friction blocks on the water cushion maintained to counteract the uplift pressure.*—The water profiles observed on the model without blocks and with block system (B) for a discharge of 300 cusecs per foot run, and different downstream water levels have been plotted in Figure 7 C.3 (*i*), (*ii*).

These show how a greater depth of water is obtained by the block system (B) to counteract the uplift pressure. This effect is very helpful when the downstream water level is low, that is, when a freshet comes suddenly at the beginning or end of the flood season when the river is practically dry.

EFFECT OF STANDING WAVE ON THE STABILITY OF THE DOWNSTREAM FLOOR AGAINST UPLIFT PRESSURE

(1) *Undersluice Section*—The designed undersluice section with sheet piles is shown in Figure 7 C.4. The floor will be built in reinforced concrete. The values of uplift pressure below the floor were worked out and are given below :—

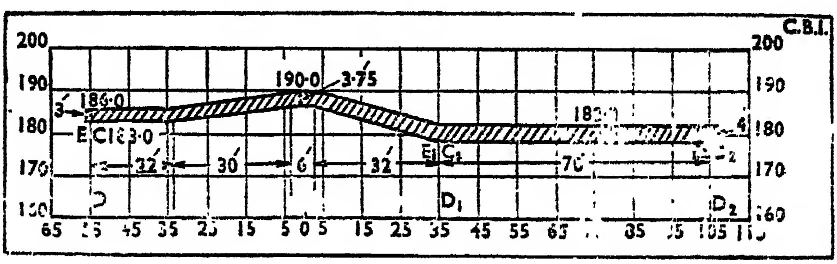

Figure 7 C.4 : *Showing undersluice section of Mor Barrage.*

Uplift pressure at C 71·2 per cent. of the total head.

Ditto.	E_1 49·6	Ditto.	Ditto.
Ditto.	C_1=42·4	Ditto.	Ditto.
Ditto.	E_2=25 0	Ditto.	Ditto.

The exit gradient works out to be GE=$\frac{1}{47}$ under the maximum head of 25 feet (=207·0 feet—182·0 feet) of water.

In Figure 7 C.5 (*i*) to (*viii*), the water profiles observed in the model for different discharges and different downstream water levels have been drawn. The corresponding hydraulic gradient lines have also been drawn. Another line PQQ′R has been drawn at a depth below the hydraulic gradient line equivalent to the depth of water that may be counter-balanced by the weight of submerged floor (specific gravity of submerged floor was taken as 1·4). If, therefore, the observed water profile falls below this line PQ′QR at any point, the uplift pressure at that point will remain unbalanced. Figures 7 C.5

discharge of 300 cusecs per foot run, both in the case of the undersluice and the weir section. If the above assumption is correct then a few points in the design of the barrage deserve consideration. The depth of the sheet pile at the downstream end of both the undersluice and weir sections (see Figure 7 C. 4 and 7 C.1) has been designed to be 16 feet below the downstream floor level. Considering the results of the above experiment, the depth should have been at-least 26 feet so as to cover the maximum depth of scour expected. The existence and extent of the unbalanced pressure on the downstream glacis discussed in paragraph 5 above also deserves attention.

TABLE 7 C.2

Mor Barrage—Weir Section—Model Scale $\frac{1}{16}$ (R.L.).

Upstream floor at 188·0 feet (R.L.); Crest at 193·7 feet (R.L.); Downstream floor at 184·0 feet (R.L.).

Upstream glacis 1 : 4; Crest width=6 feet; Downstream glacis 1 : 4.

Discharge per foot run=q cusecs	25	50	75	100	125	150	175	200	225	250	275	300
Value of K in $q=Kh^{\frac{3}{2}}$ where H is the total energy head	2·96	3·14	3·22	3·30	3·26	3·30	3·30	3·31	3·32	3·39	3.39	3.39

(2) THE LINES OF FLOW UNDER CAUSEWAYS ON SAND FOUNDATIONS (²)

The aim of the investigation was to determine the lines of sub-soil flow of water under masonry structures founded on permeable foundations like sand by utilizing the analogy between electric flow and hydraulic flow.

For the purpose of this experiment a section of the causeway on the Hyderabad=Warrangal road was chosen. It was built on sand foundation with the end walls taken to a depth of 4 feet on the upstream side and 6 feet on the downstream side. In utilizing this method the similarity of the law for the flow of water in a porous medium under a hydraulic head (Darcy's Law) and the law for the flow of the electric current in a conducting medium under difference of potential (Ohm's Law) is made use of. The equipotential lines in a conducting liquid across a potential drop with a model of the causeway (or weir) are determined. These lines represent equipressure lines in the sub-soil and the stream lines are perpendicular to this. The wooden tank in which the

(²) Hyderabad Engineering Research Laboratories, Annual Report, 1947, page 79.

experiment was conducted is 4 feet × 4 feet × 4 inches. It has a glass bottom. A $\frac{1}{24}$ scale model of the causeway made of ebonite was set up in the tank. On both sides of it were conductors of thick copper. The conducting medium in the tank was a dilute solution of Ammonium Chloride. The copper plates were connected across potentiometer. The source of alternating current used was a GE beat frequency oscillator.

Various potential drops were tapped from the potentiometer and the null points were found in the tank with the help of a Jackey moving over the tank. These points for equal drops of potential were plotted on a graph sheet and equipotential lines were drawn. Stream lines of flow were drawn by drawing squares on the equipotential lines.

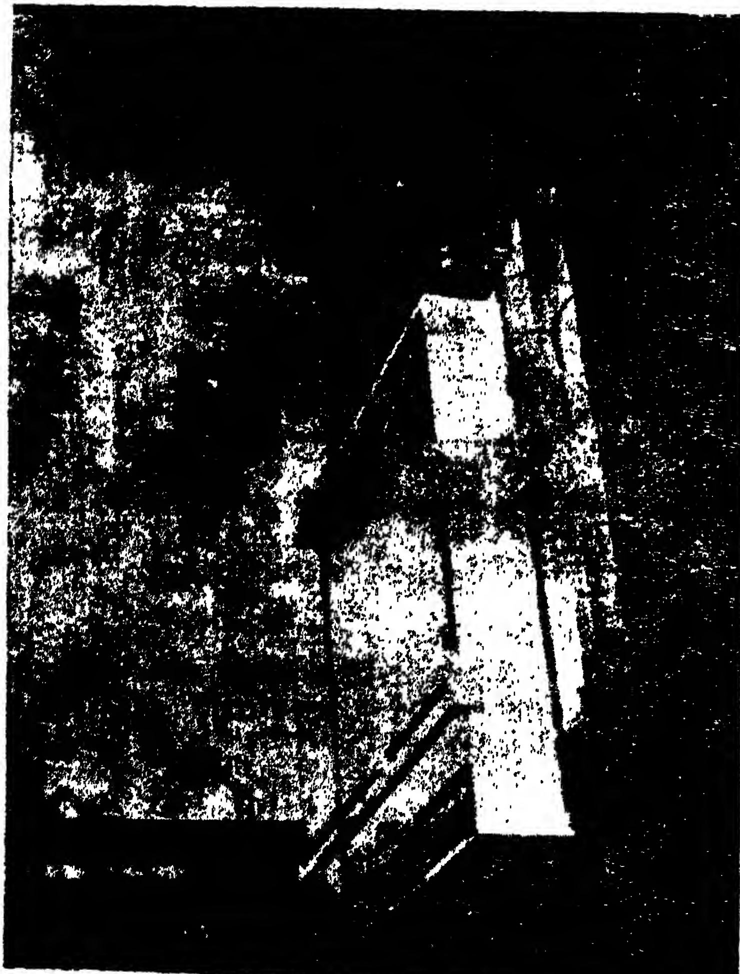

Figure 7 C.6

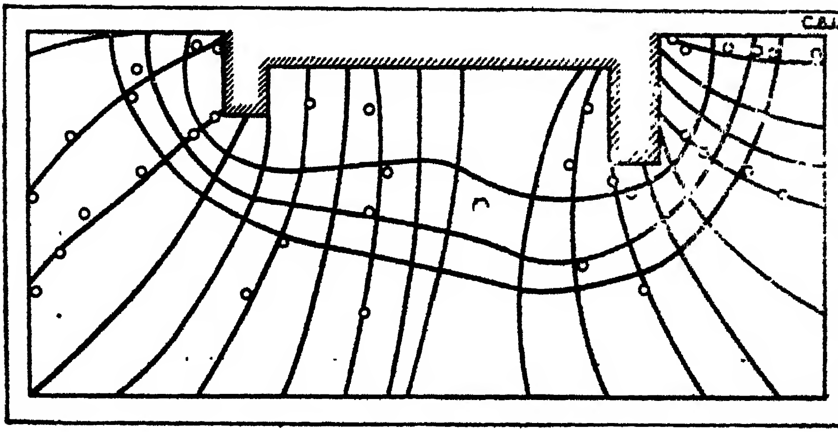

Figure 7 C.7

Figure 7 C.6 gives the arrangement of the apparatus and Figure 7 C.7 gives he lines of flow as determined by this method. This is comparable with the lines of flow obtained by injecting colour into a model of the causeway with sand foundations.

DISCUSSION BY THE RESEARCH COMMITTEE

Dr. N. K. Bose introduced item (1).

As Mr. Hardiker was not present item (2) was taken as introduced.

Rao Bahadur D. V. Joglekar commenting on item (1) said that the table in para 2 showed water levels downstream of the Barrage (*a*) before retrogression, (*b*) after retrogression of 4 feet. Rao Bahadur enquired if no accretion was anticipated.

Retrogression for all the discharges in the river ranging from 125,000 cusecs to 300,000 cusecs was assumed to be 4 feet which was incorrect. The retrogression for the maximum discharge would certainly be less than that for lower discharges.

Continuing Rao Bahadur Joglekar asked why no allowance was made for concentration of discharge intensity. The model should have been run after making an allowance for concentration till the scour stabilized.

When R.L. 208 was the retrograded water level for $q=300$ cusecs why was scour observed for water levels lower than R. L. 208·0 ?

He also enquired if the barrage was not likely to be semi-modular after accretion downstream of the barrage. The coefficient of discharge should have been worked out for these conditions also. Why was discharge co-efficient worked out under free flow conditions?

How was velocity of approach allowed for? *i.e.*, how was the value of α found in the formula $h_v = \alpha \frac{v^2}{2g}$?

Was the position of the upstream gauge relatively similar in the various models or was it kept at a constant distance from the weir irrespective of the model scale?

The co-efficients of discharge in 1/16 scale model are inconsistant with those in other scale models.

Were the co-efficients observed for various values of downstream and upstream water levels with anticipated retrogression and accretion?

As regards scour below weir, the factor $\frac{R^{3.2}}{Q}$ is stated to be not a suitable parameter for calculating the depth of scour. RAO BAHADUR JOGLEKAR considered that V^2/d (representing turbulence for producing scour) was a better parameter as explained in Annual Report, 1946 of the Central Waterways, Irrigation and Navigation Research Station.

MR. N. S. GOVINDA RAO drew attention of the Committee to Figure C-27 on page 21 of the Bengal report and explained the formula for obtaining the hydraulic gradient. In calculating uplift pressure, it might be noted that the velocity of the water flowing along the glacis had a vertical component, which also helped in counter balancing the uplift pressure.

MR. T. P. KUTTIAMMU referred to item (1) and said that for certain conditions of flow there was no standing wave formed over the *pucca* apron and consequently deep scour holes were formed. This was due to inadequate tail water depth downstream. He asked Dr. Bose to please state if there was any objection to sinking the floor and ensuring a standing wave for all flow conditions.

DR. H.L. UPPAL said that regarding the position of the downstream blocks, Mr. Govinda Rao had pointed out that if they were situated a few feet upstream the efficiency was a maximum. In 1938 Dr. Bhandari and Dr. Uppal had put up a paper on the most suitable position of the blocks, and found that the upstream row of the blocks should be at the toe of the glacis and the downstream row of the blocks at the end of the horizontal floor.

THE CHAIRMAN (RAO BAHADUR VENKATACHARI) said that he felt that silt and sand will naturally be piling up against the blocks. The chairman asked whether any upward slope had been tried on any model and the results studied.

Dr. H. L. Uppal said that at the instance of Mr. Haigh in 1944, what was called 'dishing' was experimented upon, but that made the case worse. The slope was about 1 in 10 – very gentle.

The Chairman (Rao Bahadur Venkatacharya) asked if that dishing was below the downstream bed level of the canal.

Dr. H.L. Uppal said that it was not below bed level.

The Chairman (Rao Bahadur Venkatacharya) said that in the case of concentration, all discharges from 300 down to about 100 should be worked out. In general value of 300 could not be expected without concentration. The usual flood discharge was normally about 200. A figure of 300 meant nearly 50 per cent. concentration.

Dr. N. K. Bose said that that was why investigations were carried out on the lower downstream levels. When the river was flowing uniformly it was at R. L. 208.

In reply to Mr. M. P. Mathrani Dr. Bose said that the minimum downstream water level was at R. L. 192. The downstream level was taken at about 192, because it was anticipated that the river might rise very suddenly and the extreme level may go up to 207.

Rao Bahadur D. V. Joglekar asked Dr. Bose what would happen if accretion occurred.

Dr. N. K. Bose replied that a retrogression of four feet was tried on the model The effect of accretion was not tried. The normal discharge of the river was likely to be of the order of 200 cusecs per foot run. A concentration of 50 per cent. or 300 cusecs per foot run was tried on the model. The level on the downstream side lower than 208·0 was tried to see its effect on the standing wave. Though the normal water level was 208·0, very often the flood came in the river when the downstream bed was almost dry and the upstream water level was over 206·75. The worst situation was likely to arise at that time.

Dr. Bose further said that for these experiments the position of the upstream gauge was shifted depending on the scale of the model. As a matter of fact, the water surface was observed by a pointer scale and the position of the upstream water levels was obtained from the profile.

Rao Bahadur Joglekar said that according to his suggestion the gauge should be 15 feet in this particular model. In the smaller scale it should be 10 feet.

Dr. Bose said that what Rao Bahadur Joglekar said could apply when the gauge was steady. In this case the gauge was not steady.

It was decided to keep the subject on the agenda.

DISCUSSION BY THE BOARD

THE SECRETARY said that two items were discussed at the Research Committee Meeting (page 920). There was no resolution.

DR. H. L. UPPAL wished to mention something about the variations he had observed during his experiments at Malikpur Station from the accepted theories. These investigations were by no means complete but they had yielded some astonishing results. Probably at the next Research Committee Meeting he might be able to give some definite conclusions. The main reason might be that experiments till now were conducted in 2 dimensional flow. When the third dimension is introduced one would naturally expect much change in the accepted theories.

RAI BAHADUR C. L. HANDA said that at Malikpur a model of Nangal Barrage was being constructed and their main difficulty was of reproducing the foundation which were neither permeable nor fully impermeable. These were layers of conglomerate with varying thickness interlaced with permeable stuff. Research was necessary to find out how the head was depressed under such conditions. Some definite indication as to what one should do as to permeable foundations were found quite essential and in the model under construction at Malikpur they were trying to simulate such conditions as best as possible. The limitations of such a model were obvious. One might hope to get some results of fundamental value. Originally they proceeded on the assumption that the foundation was permeable enough to reduce the weight by 50%. But when the actual excavation was done and when they encountered different layers of varying permeablity they considered that 50% was too optimistic and they should not allow more than 25%. This figure they had fixed tentatively since they could not reproduce all the conditions. But the structure had been designed as if it was founded on absolutely a permeable foundation. As for uplift no reduction is allowed and full uplift, had been considered. He hoped that this model would give some tangible results.

THE SECRETARY suggested that instead of trying to introduce conditions on the Nangal Model which were supposed to be equivalent to those on the prototype which could not be done satisfactorily would it not be better if the model was constructed for same definite known conditions. Definite results would then be obtained. Having worked on these lines later it might be possible to superimpose some conditions to represent those at Nangal.

RAI BAHADUR C. L. HANDA agreed with the Secretary. He said their model at Nangal would be no more than what Mr. Gulhati visualised. They would make systematic variation in the permeable and impermeable factors of the foundations and obtain definite results.

DR. N. K. BOSE said that he was glad that Rai Bahadur C. L. Handa had now realised that where there was a doubt regarding permeability it was better to assume the foundation to be completely porous. Speaking about the stratification of flow he said that considerable work had been done on this

subject by foreign research workers and if Rai Bahadur Handa and Dr. Uppal wanted to know about it he could supply the references. These experiments had been done with a large number of layers on scaled models. The effect observed on the flow was that the stream lines were altered. So before East Punjab started on the work, it would be better if they go through these experiments already done.

Rai Bahadur C. L. Handa observed that the conditions at Nangal were somewhat different. They were able to go as deep as 30 to 40 feet. But afterwards they were meeting with a lens of about 3 feet thick.

Dr. N. K. Bose said that the question of lens had also been dealt with by the same author in the reference he had mentioned.

Dr. H. L. Uppal said that a lot of work had been carried out during preparation days in the Punjab on the stratification of shingle and sand. The results noted at actual constructions at two or three places showed very little variation from those observed in the model.

8C. Canal Falls

PRELIMINARY NOTE

This subject came on the agenda in 1931. A Board publication was issued in 1935, which is now being revised. The progress made in this respect is rather slow for lack of staff in the Board's office.

At the 1946 Board meeting it was decided to request the Chief Engineers' Punjab to send in their final reports so that the Secretary could proceed with the proposed publication.

The contribution made at the 1946 Research Committee Meeting, it was felt, had advanced the research on this subject and Research Officers were requested to forward further contributions as early as possible.

The following items were discussed at the 1947 Research Committee Meeting :—

Experiments on one span (part model) of siphon in connection with proposed additions and alterations to the existing Tando Mastikhan fall at R.D. 119, 117 of Rohri Canal.

Canal drop with counteracting jets for the N.T. Canal.

To find a device for controlling side erosion at foot bridge at mile 8-4-232, Marihan Branch (Mirzapur Canals).

Rasul hydel scheme falls.

Fall at R.D. 129, 500, Pakpattan Canal.

Design of canal falls.

Recent Literature.

(1) Blench T., Director; Mushtaq Ahmad, Hydraulic Officer and Nazir Ahmad, Physicist, Irrigation Research Punjab—Scour in Alluvium below falls —Second meeting of the International Association for Hydraulic Structures Research, Stockholm, 1948, paper No. 4.

THE YEAR'S WORK

The following items were discussed at the 1948 meeting of the Research Committee :—

(1) Basic study on Sarda type fall.

(2) Dissipation of energy below flumed vertical falls and non flumed standing wave baffle falls.

(3) Relations between the standing wave elements.

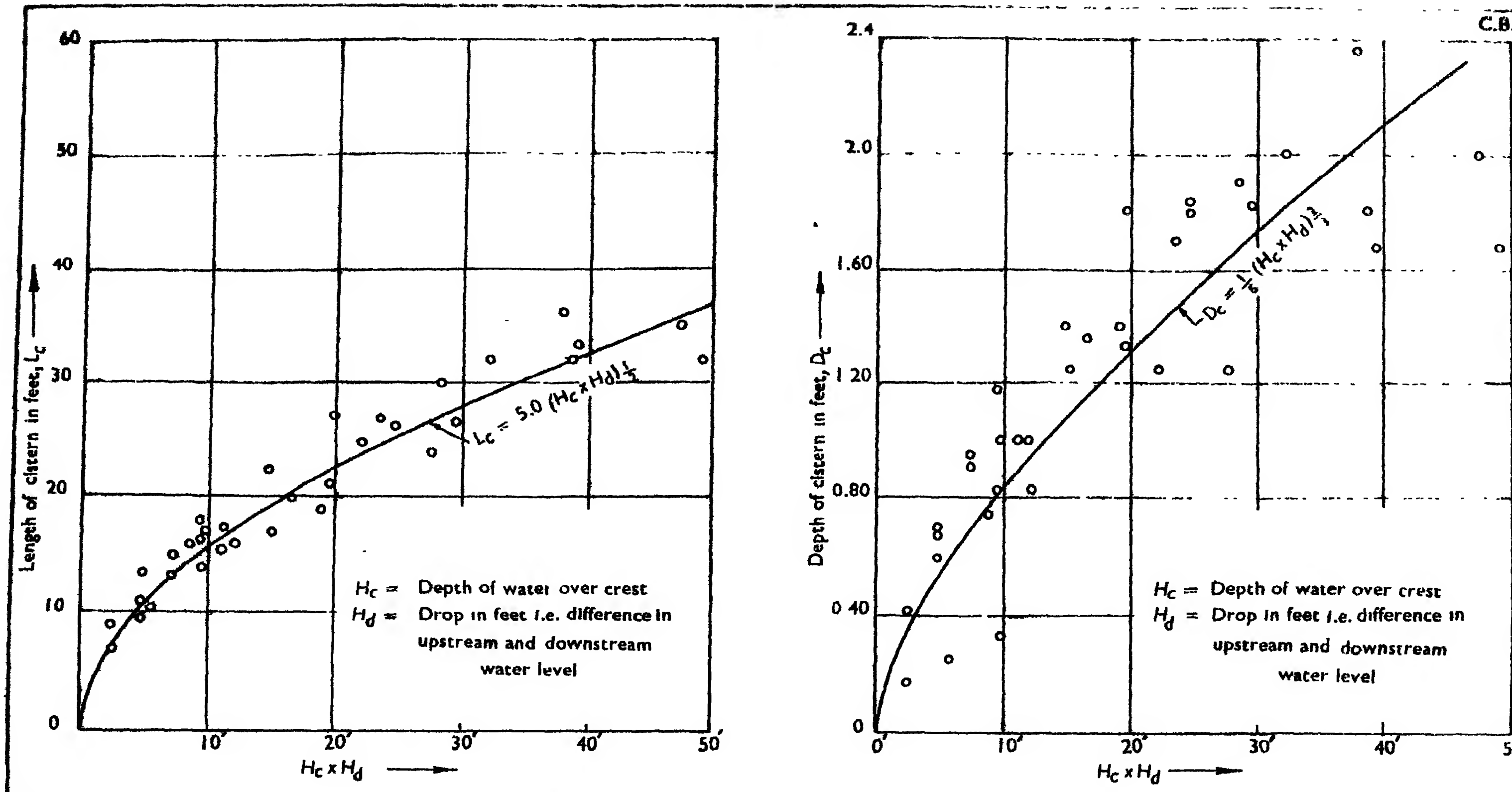

Figure 8C.1 & 8C.2:- *Showing relation of length and depth of cistern between a vertical drop fall (H_d) with discharge per foot and drop over crest (H_c)*

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(1) BASIC STUDY ON SARDA TYPE FALL (1)

ABSTRACT

The Sarda type vertical drop fall has already proved its worth as an economical, simple and efficient type of canal fall. While presenting the details of its design (2) it was mentioned about the general working of Sarda type falls that, ' at places they have developed side erosion downstream due to action of wave wash. Bed erosion is negligible except where there has been retrogression in the downstream reach of the canal. ' It was also mentioned in the same paper that efforts were being made to get over these shortcomings in the design. With that end in view, basic study on this fall was started in 1944 to evolve optimum dimensions of a cistern, which was found to be helpful both against wave wash and small amount of retrogression downstream.

Tentative formulae giving depth and length of cistern, after carrying out model investigations on a few drops and discharges per foot of the crest, were given in Technical Memorandum No. 15, Page 12 :—

Length of Cistern, $L_c = 5\,(H_c \times H_d)^{\frac{1}{2}}$

Depth of cistern $D_c = 1{\cdot}7\,(H_c \times H_d)^{\frac{1}{2}}$

Where H_c = Depth overcrest

and H_d = Drop in the water surface.

Since then the basic studies have continued and data has been collected for discharges varying from 55 cusecs to 8,000 cusecs and drops varying from 1·5 feet to 20 feet.

An attempt has again been made to deduce empirical fromulae for the optimum dimensions of cistern for different'drops' and discharges in the light of the mass of experimental data so far collected.

In these experiments, the depth of the cistern was fixed at first and the best length was experimentally arrived at, giving the minimum scour, which may be seen in column (8) of Table 8 C. 1. Having obtained the best length for a certain drop and discharge of the channel, the best depth was similarly deduced experimentally with varying depths. This is given in column (9) of Table 8 C. 1. Relationship between length and depth of cistern in terms of drop and head over crest worked out as below :—

$$L_c = 4{\cdot}697\ (H_c \times H_d)^{0{\cdot}5246}$$

$$D_c = 0{\cdot}1725\ (H_c \times H_d)^{0{\cdot}6672}$$

These formulas with a slight modification along with the experimental data are represented graphically in Figure 8 C. 1 and 8 C. 2.

(1) United Provinces Irrigation Research Station, Progress on Research during 1947, pages 73—79.

(2) United Provinces Public Works Department, Irrigation Branch Technical paper No. 7, page 5.

The quality of fit of the above formulae, as statistically tested, was found to be excellent, which is a pointer to the hypothesis that the optimum dimensions of cistern below Sarda type of fall are uniquely determined from the drop of fall and discarge per foot of the crest.

As the data collected so far fully represents all the drops and sizes of canals met in this province and as the result obtained is highly significant statistically, it is now proposed to finalise these experiments. It may, however, be mentioned that inspite of the cistern the side erosion due to wave wash is not entirely eliminated, though subdued. Provision of longitudinal vanes inside the cistern was, therefore, recommended vide Technical Memorandum No. 16, Chapter VIII, which made the side erosion downstream almost negligible. The optimum spacing of vanes of the same height as the depth of cistern is now found to be $L_{c/4}$.

TABLE 8 C. 1

Statement showing optimum dimensions of cistern below Sarda type fall experimentally obtained for various drops and discharges per foot run.

Serial No.	Drop of fall in feet	Discharge of the channel in cusecs	Discharge per foot run of crest	Depth of water over the crest in feet (H_c)	Bed width in feet	Water depth in feet	Optimum length of cistern in feet experimentally determined	Optimum depth of cistern in feet obtained for length from column (8)
1	2	3	4	5	6	7	8	9
1	2	55	4·23	1·23	13	2·6	7	0.17
2	4	55	4·23	1·23	13	2·6	10·7	0·67
3	6	55	4·23	1·23	13	2·6	13·3	0·91
4	8	55	4·23	1·23	13	2·6	16·7	0.83
5	10	55	4·23	1·23	13	2·6	16	0·83
6	3	154	7·70	1·84	20	4	10·5	0·25
7	6	154	7·70	1·84	20	4	16	1·01
8	9	154	7·70	1·84	20	4	20	1.36
9	12	154	7·70	1·84	20	4	25	1·25
10	15	154	7·70	1·84	20	4	24	1.25
11	4	500	18·79	2·46	26·6	5·3	14	0·33
12	8	500	18·79	2·46	26·6	5·3	21·3	1·33
13	12	500	18·79	2·46	26·6	5·3	26·7	1·82

TABLE 8 C. 1—*contd.*

1	2	3	4	5	6	7	8	9
14	16	500	18·79	2·46	26·6	5·3	33·3	1·67
15	20	500	18·79	2·46	26·6	5·3	32·0	1·67
16	1	1413	14·13	2·375	100	5	9	0·415
17	2	1413	14·13	2·375	100	5	9·5	0·7
18	3	1413	14·13	2·375	100	5	15	0·95
19	4	1413	14·13	2·375	100	5	18	1·18
20	5	1413	14·13	2·375	100	5	17·5	1·0
21	1·5	3840	25·6	3·3	150	7·5	13·5	0·6
22	3·0	3840	25·6	3·3	150	7·5	14·25	1·0
23	4·5	3840	25·6	3·3	150	7·5	22·5	1·4
24	6·0	3840	25·6	3·3	150	7·5	27·0	1·8
25	7·5	3840	25·6	3·3	150	7·5	26·25	1·8
26	2	6200	34·44	4·3	180	9	16	0·75
27	3·5	6200	34·44	4·3	180	9	17	1·25
28	5·5	6200	34·44	4·3	180	9	27	1·7
29	7·5	6200	34·44	4·3	180	9	32	2·0
30	9	6200	34·44	4·3	180	9	32	1·8
31	2	8000	40	4·75	200	10	18	0·83
32	4	8000	40	4·75	200	10	19	1·4
33	6	8000	40	4·75	200	10	30	1·9
34	8	8000	40	4·75	200	10	36	2·36
35	10	8000	40	4·75	200	10	35	2·0

(2) DISSIPATION OF ENERGY BELOW FLUMED VERTICAL FALLS AND NON-FLUMED STANDING WAVE BAFFLE FALLS (³)

ABSTRACT

At the 17th Annual Meeting of Research Officers of the Central Board of Irrigation, while discussing Canal Falls, Col. T. Blench, Director of the Punjab Irrigation Research Institute, suggested that, in Flumed Falls, instead of expanding the sub-critical flow below the fall, it should be better to expand

(³) Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report, Technical, 1947, pages 167-175.

the hypercritical flow above it, since the latter can be fanned out to any sharp divergence, thereby reducing the discharge intensity and leading to a cheaper design.

It is also known that vertical falls, particularly of the flumed category, have a tendency of " bellying " unless corrected by suitable devices.

There is a misguided tendency of comparing costs of flumed and unflumed falls regardless of the additional purposes served by, and the greater intensity imposed on the former. It would be useful to show that the same type of fall *e.g.*, the Central Station Baffle Fall—could be considerably cheapened if fluming is not imposed.

The present series of basic experiments was, thus, started at the Poona Research Station, with a view to—

(a) study the effects of expanding the hypercritical jet above a fall,

(b) devise means of preventing the tendency to erode the banks which cause embayments downstream of flumed vertical falls, and

(c) evolve an economical station design of a non-flumed, Baffle Type Fall.

MODELS

Geometrically similar models scale 1/6 of the flumed vertical and the non-flumed baffle type falls were designed to the following data :—

Q=100 cusecs.

H=8 feet.

d_1=d_3=3·58 feet.

B=16·20 feet.

the data being the same as that of the flumed fall designs reported in the Annual Report (Tech.) 1944.

The model bed below the fall was laid in 0·34 mm. Koregaon sand and the tests were done without any sand charge. The scour pattern obtained in the sand-bed was observed and the general flow conditions were noted and photographed to compare the designs with different modifications incorporated in them.

DISCUSSION OF RESULTS OF TESTS ON FLUMED VERTICAL FALLS

It was at once evident from the model that it is not quite correct to say that a hypercritical jet can be made to follow *any* degree of expansion, leading to uniformly reduced intensity of discharge. For example, it was observed that though the hypercitical jet followed 2 : 1 side expensions, yet the distribution of discharge across the section was not uniform and with sharper expansions the distribution was made worse. To study this factor in detail, observations of water level elevations were made with different side splays below the parallel-sided control section.

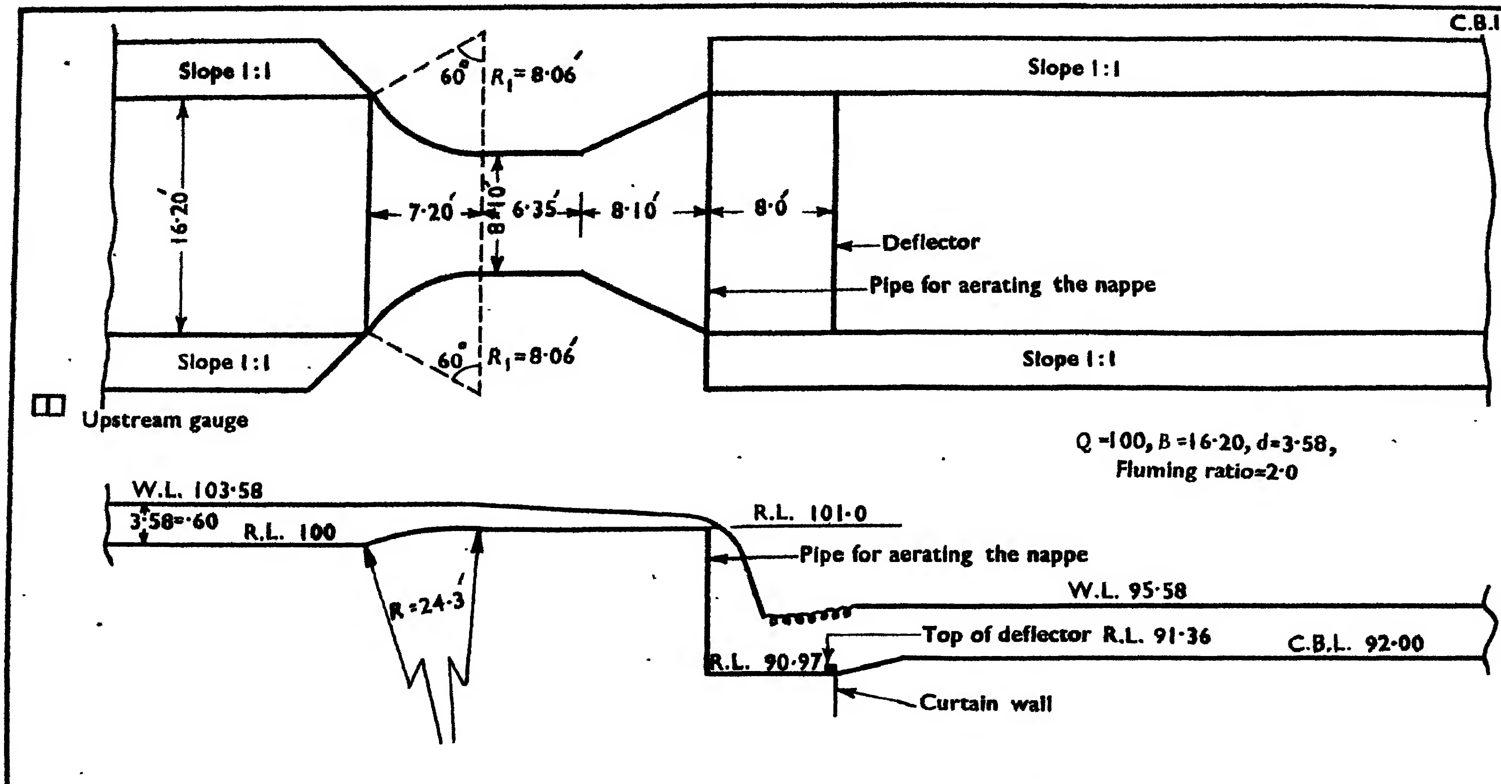
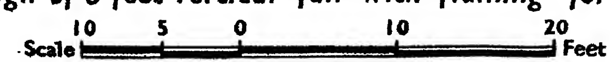


Figure 8C.3:- *Showing design of 8 feet vertical fall with fluming for Q = 100 cusecs*



Though in erodible channels, it is natural to have the discharge intensity in the centre greater than that at the side, yet just above a fall, it is desirable to get uniform distribution all across the section because this will give uniform pressure—plus—momentum required to balance the standing wave and will ultimately prevent any jetting tendency which otherwise persists due to curvature of hypercritical flow, induced by non-uniform distribution of discharge intensity.

With too low a discharge intensity at the sides above a fall, optimum dissipation of energy cannot be expected; and, the ratio of central to side depths at the end cross-section worked out to 1·74, 9·14 and 3·86 and, though the velocity in very low hypercritical depths could not be observed, yet it was noted that the discharge intensity at the sides was less than in the centre.

Comparing the discharge intensities at the centre and 6 inches (≡3 feet) from the centre line on either side in the three cases, it was seen that the effect of expansions sharper than 1 in 2 was to reduce the central intensity and *increase* it *up* to one foot *from centre*; though this happened in the central two feet strip below the throat, yet further away from the centre the discharge intensity was feeble. Generally, however, the result was that the discharge intensity all across the section was not uniform; but it was rendered less un-uniform in the *central portion* when the sides below the parallel- sided throat were made to expand along the extreme line of free flow.

On theoretical grounds, expanding sides require a rising bed along the line of flow; this was also tested but uniform discharge intensity was still not obtained.

IMPROVING THE PERFORMANCE OF FLUMED VERTICAL FALLS

Starting with the design shown in Figure 8 C. 3 the following modifications were tested to improve the scour pattern and to reduce the side embayments downstream.

(*i*) *Effect of length of cistern floor.*

It appeared that vertical falls require a relatively longer cistern floor;

(*ii*) *Depth of cistern.*

(*iii*) *Width of cistern and height of deflector.*

The condition of flow in the cistern in all the tests as observed showed that the surface flow near the sides was forward but heading towards the bank, while in the centre it was backward from the deflector towards the nappe; and, for the lower filaments, the flow indicator showed that there was forward flow in the bottom one foot depth only (out of the total depth of 4·61 feet) and that, too, occurred in intermittent spurts of fast current.

A feature, but for which the scour pattern would have been wholly satisfactory, was the bellying of the scour contour lines at the sides. **This is due to the nature of flow conditions, even in the best design so far tested.**

To ensure an ideal pattern of scour it was necessary to produce natural forward flow at the exit section of the deflector; and after a number of exploratory tests it was found that the addition of a central pier gave the desired results.

It was observed that the effect of a central pier is to pull back the flow lines, which otherwise tend to flow away from the centre, towards it. This result may be explained by the fact that due to the form drag of the pier sides, the velocities just near the pier face decrease relatively more than at quarter widths of the throat, deflecting the flow lines towards the centre.

NON-FLUMED BAFFLE FALLS

The design shown in Figure 8 C. 4 worked very well in that there was satisfactory dissipation of energy in the primary and secondary standing waves resulting in no scour downstream. From preliminary tests the pavement length was reduced to 5·58 feet $=2 \cdot 19\ (D_2 - D_1)$ and, even so, the results as regards scour and flow pattern were satisfactory.

With 25% retrogression of downstream water levels, the hydraulic jump is swept out of the stiling basins in all types of falls tested at the Station *except the Station type Baffle Fall, which works equally satisfactorily with or without retrogression.*

Exploratory experiments made so far with the Station type design for a non-proportional, *unflumed*, Baffle-type Fall have shown that 1/2 : 1 glacis and about $2 \cdot 2\ (D_2\text{-}D_1)$ feet long cistern below the baffle gives sufficient length in which turbulence is sufficiently damped down. Thus, the preliminary tests have confirmed, as is only to be expected. that where fluming is not required the Station type design can be substantially cheapened.

The difference in cost obtained by adopting a steeper glacis and shorter pavement in large falls will be appreciable

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(3) RELATIONS BETWEEN THE STANDING WAVE ELEMENTS [4]

The three main elements of a standing wave are :—

(1) Discharge per foot run$=q$

(2) Depth upstream of the wave$=D_1$, and

(3) Depth downstream of the wave$=D_2$,

From considerations of momentum and pressure, it is shown in text books that these three elements are connected by the relation :—

$$D_1\ D_2\ (D_1+D_2) = 2q^2/g \quad \ldots \quad \ldots \quad \ldots \quad (8\ C.\ 1)$$

[4] East Punjab Irrigation Research Institute, Amritsar, Annual Report, 1947, pages 39—42.

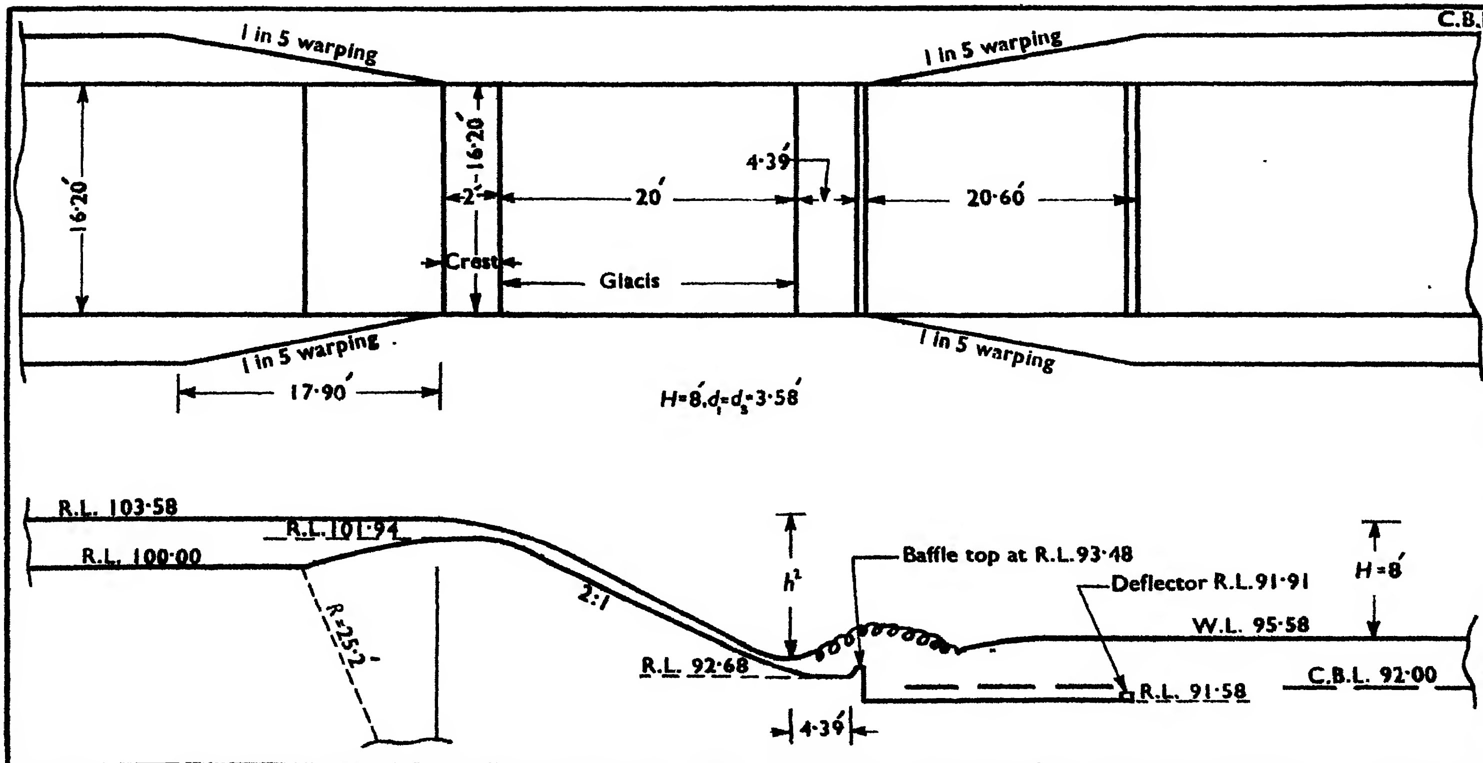

Figure 8C.4:- *Showing baffle fall design without any fluming for Q=100 cusecs*

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In standing wave problems, we also encounter the following further elements :—

(4) Energy upstream of the wave $= E_{f_1}$

(5) Energy downstream of the wave $= E_{f_2}$ and,

(6) Head lost in the wave $= H_L$

These are connected with q, D_1 and D_2, by the relations :—

$$E_{f_1} = D_1 + q/2gD_1^2 \quad \ldots \quad \ldots \quad \ldots (8\text{ C. }2)$$

$$E_{f_2} = D_2 + q/2gD_2^2 \text{ and} \quad \ldots \quad \ldots \quad \ldots (8\text{ C. }3)$$

$$H_L = E_{f_1} - ZE_{f_2} \quad \ldots \quad \ldots \quad \ldots (8\text{ C. }4)$$

There are, thus six quantities characterising the wave *viz.*, q_1, D_1, D_2 E_{f_1}, E_{f_2} and H_L; connected by four relations—(8 C. 1) to (8 C. 4). Therefore, only two out of the six can be chosen independently the others being derived, by these four relations.

The process of derivation, is, however, not equally easy in all cases. Thus we know q and D_1, we can solve for D_2 from (8C. 1) and get E_{f_1} from (8 C. 2); the substitution for D_2 in (8 C. 3) gives E_{f_2} and then H_L is got by simple subtraction But if we know q and H_L, the algebraic solution fom D_1, D_2, Ef_1 and Ef_2 is almost impossible.

To get over the difficulty , the procedure is :—

(*i*) For a given value of q, give D_1 different values and find the corresponding values of D_2 by solving (8 C. 1).

(*ii*) Find E_{f_1} and E_{f_2} by substituting in (8 C. 2) and (8 C. 3) and then H_L from (8 C. 4).

(*iii*) Plot H_L against the element which is to be evaluated say D_2.

We thus get a curve connecting D_2 with H_L, for a given value of q. By giving other values to q, we could get other similar curves ; so that finally there is a family of curves representing the relation between D_2 and H_L for specific values of q.

If, in a given problem, we know q and H_L, we can choose the curve which gives the relation between H_L and D_2 for the known q and read, from it, the Value of D_2 corresponding to the know H_L

Crump's idea of 'non-dimensional' ratios was calculated to simplify the calculations for such curve-families. Briefly, he took the 'critical' depth D_c,

which is a direct function of q by virtue of the relation.

$$D_c=(q^2/g)^{\frac{1}{3}} \quad \ldots \quad \ldots \quad \ldots \text{(8 C. 5)}$$

as a base, and expressed the ratios of D_1, D_2 and H_L to it by the symbols x, y and z respectively. As D_1, D_2 and D_c all had the dimensions of length, x, y and z were 'non-dimensional' ratios.

Later, the ratios of E_{f_1} and E_{f_2} to Dc were expressed by the symbols m and n.

Crump showed that x, y, z, m and n were connected by the relations :—

$$xy\,(x+y)=2 \quad \ldots \quad \ldots \quad \ldots \text{(8 C. 6)}$$

$$x+\tfrac{1}{2}\times x^2=m \quad \ldots \quad \ldots \quad \ldots \text{(8 C. 7)}$$

$$y+\tfrac{1}{2}y^2=n \quad \ldots \quad \ldots \quad \ldots \text{(8 C. 8)}$$

$$\text{and } z\,(m-n)=(y-x)^3/4xy \quad \ldots \quad \ldots \quad \ldots \text{(8 C. 9)}$$

The question of explicity showing that all the ratios were interconnected, however, still remained. This was tackled in the Institute as follows :—

Let $x=t-u$

$y=t+u$

Substituting in (8 C. 6) we get

$$2t\,(t^2-u^2)=2$$

or $$u^2=t^2-\frac{1}{t}$$

i. e. $$u=\left(t^2-\frac{1}{t}\right)^{\frac{1}{2}} \quad \ldots \quad \ldots \quad \ldots \text{(8 C. 10)}$$

$$x=t-\left(t^2-\frac{1}{t}\right)^{\frac{1}{2}} \quad \ldots \quad \ldots \text{(8 C. 11)}$$

and $$y=t+\left(t^2-\frac{1}{t}\right)^{\frac{1}{2}} \quad \ldots \quad \ldots \text{(8 C. 12)}$$

Hence $$m=x+\tfrac{1}{2}x^2 \quad \ldots \quad \ldots \quad \ldots$$

$$=t-\left(t^2-\frac{1}{t}\right)^{\frac{1}{2}}+t^2\left(t-\left(t^2-\frac{1}{t}\right)^{\frac{1}{2}}\right)^{\frac{1}{2}}$$

$$=t+t^2\left(t^2-\frac{1}{t}\right)^{\frac{1}{2}}+\frac{t}{2}-\left(t^2-\frac{1}{t}\right)^{\frac{1}{2}}$$

$$=t^4+0\cdot5t+t\left(t^2-\frac{1}{t^2}\right)^{\frac{3}{2}} \quad .. \quad .. \text{(8 C. 13)}$$

$$\text{Similarly } n=t^4+0\cdot5t-t\left(t^2-\frac{1}{t}\right)^{\frac{3}{2}} \quad .. \quad .. \text{(8 C. 14)}$$

$$\text{and } z=m-n=2t\left(t^2-\frac{1}{t}\right)^{\frac{3}{2}} \quad .. \quad .. \text{(8 C. 15)}$$

Equations (8 C. 11) to (8 C. 15) showed that the *five non-dimensional ratios x, y, z, m and n were single valued functions of the parameter, t.* The symmetry between the expressions for x and y and for m and n was also noticeable.

It followed that each of the ratios was a single-valued function of the other four.

All the ratios were thus expressed, for the first time, in terms of a single parameter.

The next step attempted was to use this parametric representation for evaluating each of the ratios in terms of the others. This was not very successful as the reversal of series involved in expressing t in terms of x or y from equations (8 C. 11) or (8 C. 12) did not yield an easily convergent expression. Further attempts were, however, in hand at the close of the year and the problem has not been altogether given up.

The object, of course, is to find, if possible, simple approximations for each ratio in terms of the others, so that the standing wave elements may be easily evaluated in terms of one another.

Another line which was pursued was the evaluation of the corresponding values of x, y, z, m, and n. At first, the method used was to give different values to the parameter t, *viz.* 1·00, 1·01, 1·02, ... 3·00, and to obtain the corresponding values of the five ratios. These calculations were, however, temporarily given up in favour of the more direct evaluation of y, z, m, n in terms of x. The convenience in this case is that as x must lie between 0·0 and 1·0, the calculations are confined to a definite range.

During the year, calculations were completed for values of y, z, m and n for x=0·001, 0·002, 0·003..........200.

The rest of the range from 0·201 to 1·000 will be completed next year.

It is proposed to publish the resulting tables in a suitable form ; as they will be the most extensive worked out so far. Their application to the design of weirs, falls and outlets will also be studied.

Another allied problem is the extension or adaptation of the curves in Plates XI.1 and XI. 2 of the Central Board of Irrigation Publication No. 12 ' Design of weirs on permeable foundations ', to the design of dams, where the values of H_L

are very much bigger than those involved in weir design. This problem had been tackled towards the close of the year and a solution appeared in sight, but the working out of details had to be postponed due to more urgent work.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. S. N. GUPTA introduced item (1) and said that basic experiment had been in progress in the United Provinces to standardise the design of Sarda type fall for the last few years. Previous investigations indicated the necessity of a cistern to be provided below the vertical drop for the maximum dissipation of energy and to eliminate the effect of wave wash.

Observations were taken in the case of various falls for drops ranging from 1·5 feet to 20·0 feet, and discharges varying from 55 cusecs to 8,000 cusecs to determine the optimum dimentions of the cistern. The statistical analysis of the data showed that :

$$L_c = 5\,(H_c . H_d)^{\frac{1}{2}}$$

$$D_c = \tfrac{1}{6}\,(H_e . H_c)^{\frac{2}{3}}$$

where L_c=length of the cistern, D_c= depth of the cistern, H_c=depth over crest, and H_D=drop in the water surface.

The effect of wave wash still persisted. Further experiments indicated that longitudinal vanes provided in the cistern having a spacing $\frac{L}{4}$ reduced the wave wash considerably, but the results were not yet finalised.

Referring to item (2) MR. GUPTA said that it was interesting to note the results of Central Research Station, Poona for a flumed vertical fall. On page 171 of their Annual Research Report, the length of the cistern as indicated by their model of 8·0 feet fall, worked out to be 7·5 $(D_2—D_1)$ or 25·0 feet whereas by the United Provinces formula it came to 26·70 feet, indicating high degree of similarity in their action of falling water in the case of flumed type and crest type vertical falls. The depth of the cistern, however, was not the same in the two cases; being greater in the Poona fall. This was probably, due to the fact that the cistern depth had invariably been adopted in accordance with the formula given in Bombay P.W.D. hand book (1925 edition) and in addition a suitable deflector had worked well in a deeper cistern.

MR. S. T. GHOTANKAR introduced item (2) and said that these experiments were started to test Col. Blench's idea to expand the hyper-critical jet above the fall rather than below the fall, as the cost of structure would be less, because a hyper-critical jet can follow a very sharp expansion. Side by side it was also tested if the 'bellying out' tendency below vertical falls could be prevented.

Experiments were also done with a *non-flumed baffle* fall with the object of reducing the cost. The *flumed* baffle falls evolved at the Poona Station had been wrongly compared with other *non-flumed* falls. The results obtained were very satisfactory.

Dr. J. K. Malhotra introduced item (3) and said that the revision of the Central Board of Irrigation Publication No. 4 on "Hydraulic Diagrams" was considered by the C.B.I. in 1946 and it was found that the calculations from which the original diagrams were compiled were not available.

The Secretary, Central Board of Irrigation, therefore, requested Mr Montagu, the original compiler of the diagrams and then Chief Engineer Punjab for help who passed on the job to the Irrigation Research Institute, Lahore. As Dr. Malhotra was interested in this work, these calculations were assigned to him and were completed by him early in 1947. During the process he tried to simplify the formula involved, and though he did not succeed in his object he got some interesting bye-products. For example, he was able to express all Crump's non-dimensional ratios in terms of a single parameter, which Crump had failed to do. Dr. Malhotra wrote down the following equations by Crump :—

$$xy\,(x+y) = 2$$

$$z = (y-x)^{\frac{3}{4}xy}$$

$$m = x+\tfrac{1}{2}x^2$$

$$n = y+\frac{1}{2y^2}$$

and said that obviously x, y, z, m and n were inter-connected, but this had not been explicitly proved so far. By introducing a parameter, t, he had however, got the equations :—

$$x = t-\left(t^2-\frac{1}{t}\right)^{\frac{1}{2}}$$

$$y = t-\left(t^2-\frac{1}{t}\right)^{\frac{1}{2}}$$

$$z = 2t\left(t^2-\frac{1}{t}\right)^{\frac{3}{2}}$$

$$m = t^4+\frac{t}{2}+t\left(t^2-\frac{1}{t}\right)$$

$$n = t^4+\frac{t}{2}-t\left(t^2-\frac{1}{t}\right)^{\frac{3}{2}}$$

which achieved this objective.

His next idea was to link up these ratios through some sort of simple formulae. He had not been very successful in his attempts to get the ratios as simple functions of each other but he was still trying and hoping that he would be able to get something out of it.

Continuing DR. MALHOTRA said that recently he had some enquiries as to how to work out the elements of a standing wave given the values of head lost, H_L, and discharge intensity, q. He pointed out that where plates XI (1) and XI (2) of the C.B.I. Publication No. 12 ' Design of weirs on permeable foundations ' would normally serve for values of H_L upto 20 feet and of p upto 400 cusecs per foot run, it was possible to use these very same curves for higher values as well, by a very simple method, which he then briefly explained.

Referring to item (1) DR. MALHOTRA said that in table No. 22 of the United Provinces Report on which the formulae for the optimum length and depth of cisterns was based, it seemed to him that some of the experiments were based on geometrically similar models, which reduced the number of independent observations from 35 to 15. The correlation might, therefore, need to be re-checked.

RAI BAHADUR S.D. KHANGAR referred to item (2) and asked what the position of the baffle platform was with reference to the canal bed level.

RAO BAHADUR D.V. JOGLEKAR replied that the position of the baffle platform was determined by the discharge and the difference of water levels upstream and downstream and was independent of the downstream canal bed. In fact it might work out to be fresh at higher, equal or lower level than the canal bed level. This position was such that a standing wave would naturally form at the toe of the fall without the aid of the baffle.

It was decided to keep the subject on the agenda.

DISCUSSION BY THE BOARD

THE SECRETARY said that three items were discussed at the Research Commitee meeting (page 934). There was no resolution. As regards Boards, revised publication on the subject, the work could not be undertaken in the Boards office for want of staff.

DR. N. K. BOSE said that a sub-committee had been appointed to review the draft of the revised publication " Standing Wave or Hydraulic Jump " prepared in the Boards' office by Dr. Bhandari. The Sub-Committee comprising Rai Bahadur Kanwar Sain, Dr. N. K. Bose, Rai Bahadur C. L. Handa and Dr. J.K. Malhotra met on 6th, 7th and 8th December, 1948. Mr. T. P. Kuttiammu was unavoidably absent.

The Sub-Committee had gone over the text in detail. They wished to record their appreciation of the great amount of reading and labour put in by Dr. Bhandari in preparing the draft. In view of its being an altogether new and vastly improved version the Sub-Committee suggested that this fact should be recorded in a suitable form in the publication itself.

The Sub-Committee discussed and recorded a number of suggestions for adding to the utility of the publication. The additions and alterations would accordingly be carried out in the text by Dr. Bhandari and a complete draft prepared, if possible, by the time of next Winter Meeting of the Research Committee, so that the final draft would be available for consideration at the next Research Committee Meeting in 1949. In view of the large number of references required to be consulted in this connection, the Sub-Committee suggested that Dr. Bhandari be given all the help necessary to enable him to devote his attention to this very important draft.

On behalf of the Board the Secretary thanked the members of the Sub-Committee for the work they had put in on the draft. He was glad to learn that the draft had their general approval. Their recommendations would no doubt be incorporated in the revised draft which would be available as early as possible.

9C. Distributary Heads

PRELIMINARY NOTE

This subject has been on the agenda since 1940, and has been considered from three aspects, *viz.*, regulation, metering of supply and silt control.

At the 1945 meeting of the Board it was resolved that a Central Board of Irrigation publication should be issued on the "Design of distributary heads" and with this end in view research officers be asked to contribute their drafts on the subject by May 31, 1946.

The Punjab, Sind and Madras have sent in their contributions. Central Waterways, Irrigation and Navigation Research Station suggested last year that certain publications were for inclusion on the subject as their contribution. Reply from United Provinces is still awaited. The rest of the Provinces and States have nothing to contribute. This publication is now ready for compilation.

The following items were discussed at the 1947 Research Committee Meeting :—

(1) Basic experiments to investigate the effect on silt distribution of

(a) angle of offtake

(b) varying discharges

(2) To find a suitable device for eliminating bed scour below Ghaggar Canal Head.

(3) Basic experiments for determining co-efficients for radial gates.

(4) Model experiments in connection with scour below cross regulator over Sind Canal at Rustom.

(5) Model experiments in connection with silting of Saidkhan distributary ex-Hydrabad Branch.

(6) Model experiments in connection with silting of Dalore and Patci distributaries ex-Jamrao Canal.

(7) Model experiments in connection with silting of Jamrao Canal,

THE YEAR'S WORK

DISCUSSION BY THE RESEARCH COMMITTEE

There was no contribution on the subject and no discussion took place at the 1948 meeting of the Research Committee.

DISCUSSION BY THE BOARD

THE SECRETARY said that there was no contribution and no discussion at the Research Committee Meeting. As regards proposed publication of the Board on the subject, the only note awaited was from United Provinces.

MR. GUPTA said that he would be sending the U. P. contribution in a month's time.

10C. Excluders and Ejectors

PRELIMINARY NOTE

This subject has been on the agenda since 1938 and papers have been contributed describing the design and the results of excluders and ejectors in the Punjab dealing with various aspects of design and giving experimental results.

At the 1945 meeting of the Board it was resolved that a Central Board of Irrigation publication should be issued on the subject summarising the position upto 1945-46, and that the Punjab Irrigation Research Institute, Lahore, be requested to draft the proposed publication.

The Chief Engineer, Punjab Irrigation, issued instructions to the Director, Irrigation Research Institute, Lahore to prepare a draft of the proposed publication.

The following items were discussed at the 1947 Research Committee Meeting.

(1) Model experiments for exclusion of silt from Damodar Canal.

(2) Analysis of observations on silt exclusion at the headworks of Damodar Canal at Rhondia.

(3) Silt exclusion works at Lloyd Barrage at Sukkar—by T. R. Sethna.

(4) Design of Lower Sind Barrage at Kotri for effective sand exclusion from canals—Sind.

(5) Optimum shape of the nose of Keti Island at Kotri for the Lower Sind Barrage.

(6) Model experiments on the exclusion of sand from a feeder channel taking off from a river.

(7) Sediment investigations—river Sutlej at Hussainiwala headworks.

(8) Sediment investigations—river Yamuna at Tajewala headworks.

(9) Sediment investigations—river Ravi at Dhanna weir.

THE YEAR'S WORK

The following items were discussed at the 1948 Research Committee Meeting :—

To design a suitable type of shingle excluder for the new supply channel at Bhimgoda weir, Hardwar.

Damodar River Model.

Silt exclusion experiment at the headworks of the Damodar Canal at Rhondia.

Comparison of sand exclusion at the nose of the outer bank and the raised sill of the approach channel under present conditions and with the outer bank extended to position Q—experiments in 1/80 : 1/40 part width Sukkur Barrage Model.

Effect of change in regulation operations of the right pocket on sand exclusion—experiments in 1/80 : 1/40 part width Sukkur Barrage Model.

Supply of sand free water to Jamshedpur Powerhouse.

Model of Nangal Canal excluder.

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(1) TO DESIGN A SUITABLE TYPE OF SHINGLE EXCLUDER FOR THE NEW SUPPLY CHANNEL AT BHIMGODA WEIR, HARDWAR [1]

ABSTRACT

There are two principal feeders to the Ganga Canal at present, known as old and new supply channels. The present Bhimgoda Head works were constructed in 1920 before which the old supply channels upstream used to be fed by temporary *bunds* after every monsoon. The main stream upstream before 1920 used to be on the left. This naturally caused considerable difficulty after the construction of the present head-works in feeding the new supply channel during the winter. The new supply channel was also receiving lot of shingle when fed during rains. To get over this difficulty a new channel known as B. B. cut was developed at right angles to the sluices which ultimatly became the main stream.

As the off-take usually takes bigger silt charge than its share, the entire charge of the river started passing through the B. B. cut, so the shoaling tendency of the new supply channel was gradually aggravated.

Another contributary factor for shoaling the new supply channel, which also slightly helped in causing increased retrogression downstream of the sluices discussed earlier, was the raising of cill level below sluices by one foot *i.e.*, upto R.L. 939 carried out in 1944. It was decided to study the causes of increased shoaling in the new supply channel and suggest preventive measures.

POSSIBLE PREVENTIVE MEASURES

From the model of the River Ganga referred to in connection with flood protection of Kankhal it was also clear that the new supply channel was taking considerable bed load at almost all the supplies when there was bed

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[1] United Provinces Irrigation Research Station, Report on Research Progress, during 1947, Pages 80—84.

movement. However, at low floods of the range of 50,000 to 90,000 cusecs, the shoaling was found to be very much increased as a result of regulation. But this aspect of the problem has not been studied in greater detail so far, nor would this alone matter much. The conditions obtained at site suggested very strongly a shingle excluder, as—

(*i*) The approach channel is straight in front of the sluices.

(*ii*) Any amount of escapage is possible for exclusion.

(*iii*) No bed movement takes place below 20,000 cusecs and so no escapage required except when the river is in floods.

(*iv*) Available use of regulating gate in bay No. 1 (width=50 feet) for creating difference of head at the entrance of tunnels.

(*v*) Plenty of drop available at sluices at all floods.

MODEL INVESTIGATIONS

A part model consisting of the sluices, and bays No. 1 and 2 of the weir was constructed and laid in Ranipur and with a 10% mixture of shingle varying from 1/8 inch to 3/8 inch.

The scales adopted were :—

Length scale $L_r=\frac{1}{40}$

Depth scale $D_r=\frac{1}{24}$

Discharge scale $Q_r=\frac{1}{5,000}=L_r\,D_r^{\frac{3}{2}}$

Time scale $T_r=\frac{1}{96}$

After proving the model, two observation pits were built in the supply channel downstream of the head regulator. With the running of flood cycle for 1947 the shoaling in the supply channel reached up to R.L. 946, *i.e.*, about three feet higher than the crest of the head regulator. The cubical contents of the shingle deposit within the observation pit was found to be 15·15 cubic feet.

Next four bell-mouthed excluder tunnels of size 11 feet×4·25 feet were constructed to cover lines of flow towards 3, 2, 2 and 3 bays of the head regulator from the upstream. Floor of the tunnels was kept at R. L. 938 and exit at R. L. 939, the cill level of the sluices, with a smooth slope upwards at the exit. This did not prove at all satisfactory as there was considerable turbulence at the entrance of the tunnels due to excessive bell-mouthing.

In the next experiment, straight openings were provided in place of bell-mouthed entrances for the tunnels and the exit lowered to R.L. 938 by removing the one foot cill. This resulted in about 60% efficiency, there being only 3·8 cubic feet of shingle deposit in the observation pit. Keeping the size of the tunnels as 11 feet×4·25 feet, a slight curvature was provided at the face which resulted in increasing the entrance width to 25 feet and the neck to 13·5

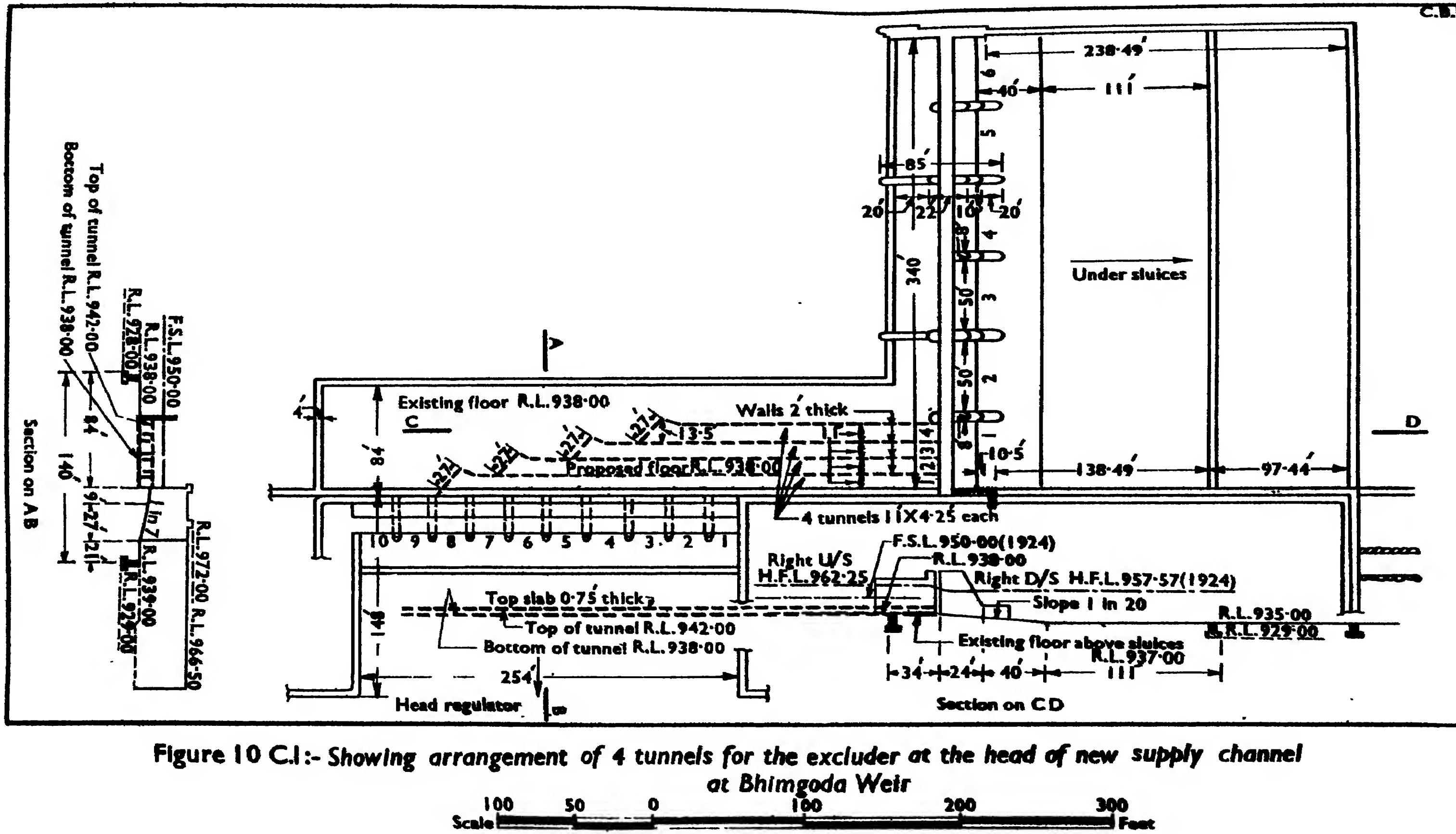

Figure 10 C.I:- *Showing arrangement of 4 tunnels for the excluder at the head of new supply channel at Bhimgoda Weir*

Reg. No. 2737 XDD (C)' 50 - 1,210 (P.Z.O.).

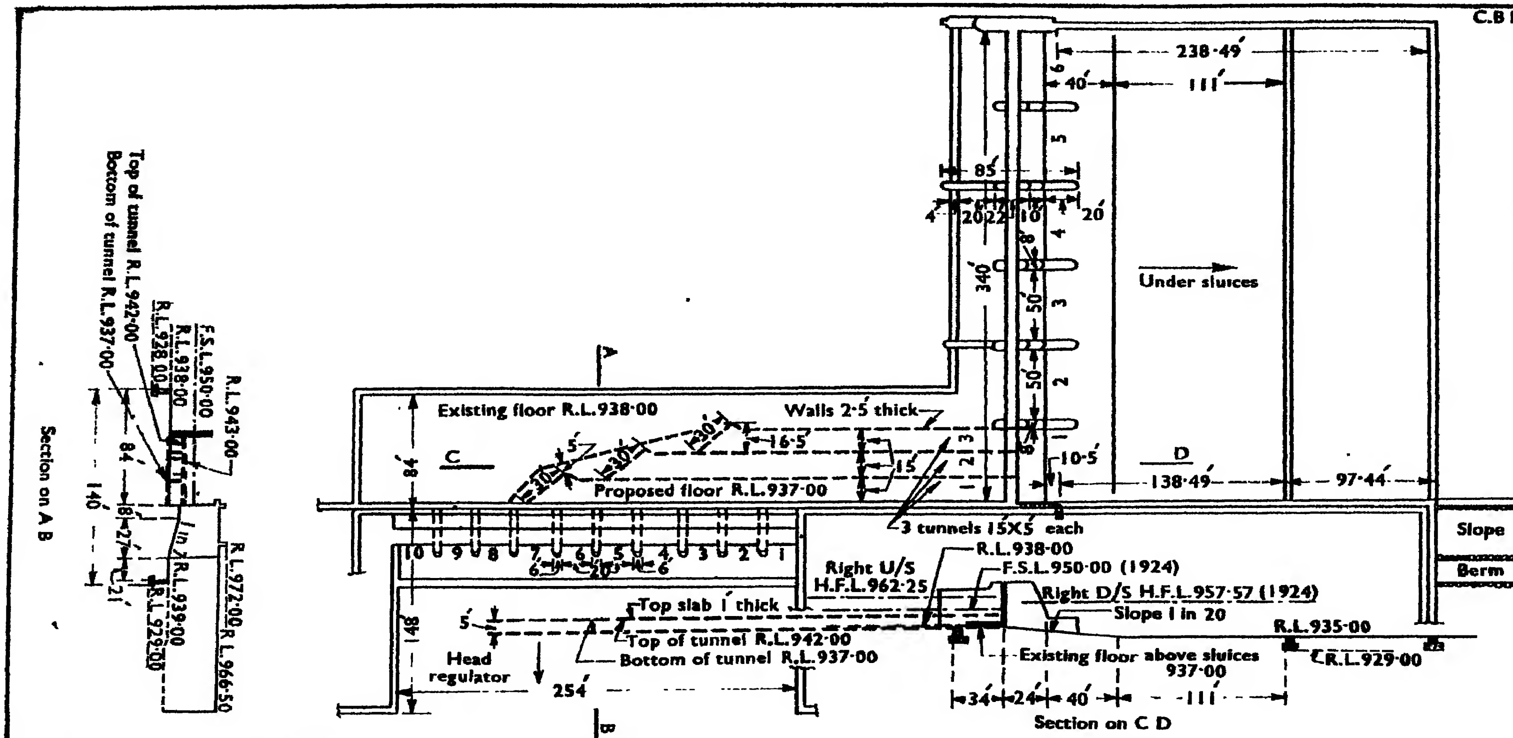

Figure 10.C2:- *Showing arrangement of 3 tunnels for the excluder at the head of new supply channel at Bhimgoda Weir*

Scale 100 50 0 100 200 300 Feet

Reg. No 2737 XDD (C)' 50 - 1,210 (P Z.O.).

feet. In this series of experiments, several positions were tried to obtain the best arrangement of the tunnels to cover the lines of flow entering the supply channel. The arrangement as per Figure 10 C. 1 was finally decided for a four tunnel silt excluder. It was, however, noticed that at high floods when the standing wave was pulled up, there was a tendency of accumulation of shingle on the floor downstream of the tunnels. This could, however, be easily washed away by raising the gate of sluice No. 1 and thus increasing the discharge through the sluice. The efficiency of this excluder worked out to 87 per cent.

It was observed in the above design that the first tunnel was usually extracting less bed charge as the bed currents in front of this tunnel were deflected away due to curved upstream wing of the regulator. In view of this and apparently greater advantage in having a bigger sized tunnel, it was considered desirable to test another design with three tunnels as per Figure 10 C. 2 keeping the tunnel dimensions 15 feet × 5 feet. The Executive Engineer, Northern Division, told us that the bed of these tunnels would require granite flagging. Since the existing floor in front of the regulator had to be uprooted in any case, it was decided to lower the bed of the tunnels to R.L. 937. This would also give better working head. On running the model, it was found to make much less deposits on the floor downstream as the hyper-critical jet issuing from the tunnels was lower in comparison to the jet from sluice No 2 which fanned out before the tunnels and automatically helped in removing all the excluded material. The design was also found to be more efficient than the design of Figure 10 C. 1. The bigger tunnels provided greater space within. and in consequence, less frictional losses. They are preferred, their chances of getting choked or blocked up with floating debris are more remote. The model was further tried for higher floods up to 6 lakhs (30 cusecs in the model) and the general efficiency was not found to be affected at any stage.

RECOMMENDATIONS

(1) **Tunnel excluders as per Figure 10 C. 2 are to be built for preventing the shoaling of the new supply channel.**

(2) The gate of bay No. 1 is to be kept lowered up to the top R. L. of the tunnels or lower if need be at very low floods whenever it is possible to escape water. Maximum difference of head between the entrance and exit of the tunnels should be created by regulation as far as possible.

(3) If accumulation of shingle is noticed below sluice No. 1, the chances of which are extremely remote, the gate may be momentarily raised to flush it out.

(4) Care may be taken to prevent the drift wood getting too near and entering the mouths of the tunnels.

(5) At very high floods if the gate in sluice No. 1 is likely to be overtopped, it may be raised if desired only to the extent that it is prevented from being overtopped. This will continue to impose adequate difference of head at the mouth of tunnels.

(6) As the boulders are expected to be sucked in with good speed, the noses of the divide walls between tunnels may be provided with angle iron casing and the floor of the tunnel flagged with granite.

(7) The river is recommended to be spread out by suitably lowering the crests of the weir so that the intensity of bed material brought down to the sluices is reduced.

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(2) DAMODAR RIVER MODEL ([2])

ABSTRACT

This model was undertaken to study the following two problems :—

(1) Exclusion of silt from the Damodar Canal.

(2) Prevention of erosion at the upstream nose of the right bank Bell *Bund* of the Anderson Weir opposite the Damodar Canal Head Regulator.

EXCLUSION OF SILT FROM THE DAMODAR CANAL

As explained on page 82 of 1946 Annual Report, methods of silt exclusion by judicious gate manipulation only were tried in the model. The canal takes off from the left undersluice pocket. There are three bays (each 60 feet wide) in the undersluice and 5 vents (each 20 feet wide) in the head regulator of the canal. The normal working procedure is to keep the three undersluice bay openings equal and so also for the head regulator vents. The top of the undersluice gates is at the level of the crest of the weir, when they are fully lowered on the undersluice floor to prevent undershot flow. These gates are designed to stand when submerged, only one foot of water above their top levels, so that they are to be raised as soon as the depth of water above their top level exceeds one foot. The crest level of the weir as well as the top of the undersluice gates when fully lowered is 164·00 (R.L.). The supply into the canal is allowed so long as the

([2]) River Research Institute, West Bengal, Annual Report, 1947, Pages 97—99.

water level in the undersluice pocket is below 170·0 (R.L.). The still pond system cannot, therefore, be practised with the existing undersluice gates. The only thing that can be done is to have the undersluice bay openings unequal instead of equal, as usually practised. The following three arrangements of undersluice bay openings were tried in the model to see if the silt entry into the canal shows any difference in the three cases :—

(1) Openings increasing from left to right.

(2) Openings equal.

(3) Openings decreasing from left to right.

The openings in the head regulator vents were kept equal in all the three cases. In each case a steady discharge of 100.000 cusecs was passed over the model and proper water-level maintained upstream and downstream of the weir. A discharge equivalent to 2 000 cusecs in the prototype was allowed into the canal by proper adjustment of head regulator gates. The proper openings in the undersluice bays in the above three cases were found in the model to be—

(1) Left bay—1¼ inch. Middle bay—1¾ inch. Right bay—2¼ inch.

(2) Left bay—1¾ inch, Middle bay—1¾ inch, Right bay—1¾ inch.

(3) Left bay—2¼ inch, Middle bay—1¾ inch, Right bay—1¼ inch.

Sand was dropped at the cross-section one mile above the weir near the left bank at a rate so that the river can just carry the amount forward and there is no deposition at the place of dropping. The model was run for six hours in each experiment, while 0·9 c. ft. of sand was dropped as stated above and the amount entering the canal during the course of the experiment was measured at the end of the run. The results were not consistent. The same experiments repeated did not give similar results regarding silt entry into the canal. There may be various reasons for such inconsistency but one of the reasons is surely this that the state of wetness of the model bed (moisture content), before water is first let over it, cannot be similar in two different experiments.

Attempts were made by dropping small wax balls made just heavier than water by admixture of chalk powder to find out the difference, if any, of silt entry into the canal in the three types of undersluice bay openings. But due to the difficulty in producing large number of these balls in uniform size, sufficiently small, the idea was abandoned and mustard seeds were tried as substitute. Mustard seeds were first put into water and allowed to stand for an hour. The portion floating on the surface was removed and the remaining portion was properly washed by water. The required quantity of this washed sample was then weighed when just moist and dropped in the model at the cross-section ½ mile above the weir during a fixed period of time, the model running

steadily during this period and an hour more with discharge 100,000 cusecs. The seeds entering the canal during this period were trapped and weighed after the experiments ; weighing being done in a similar moist stage as before. It could not, however, be guaranteed that the moisture content in the two stages are exactly equal. Here also the results are not very consistent. But as the moisture content of the seeds might have been different at the time of weighing in the different experiments, it will not be safe to draw any definite conclusion by comparing the weights of the seeds entering the canal. A better procedure would be to count the number of seeds entering the canal in each experiment and to use the seeds of the same variety in different experiments to be compared.

(3) SILT EXCLUSION EXPERIMENT AT THE HEADWORKS OF THE DAMODAR CANAL AT RHONDIA [3]

The Damodar Canal takes off from the River Damodar at Rhondia. The weir at Rhondia has a fixed crest at 164·0 R.L. of a total length of 3,750 feet and an undersluice portion with a flat sloping floor at R.L. 156·0 at the centre line of the weir. There are 3 gates each 60 feet wide in the 3 bays of the undersluice. These gates open from the above, the top level of the gates when silting on the floor being 164·0. These gates are generally opened uniformly to maintain a certain pond level and the canal regulator gates (five in number, each 20 feet wide) are also opened uniformly to admit the desired discharge into the canal.

The canal runs through high cutting in the first 2 to 3 miles of its head reach and the spoils had also been thrown on the berms during construction. During rains these spoils slide down to the bed of the canal and create a sort of a bottle neck at mile 2 of the canal. In consequence, the slope of the canal flow is flattened and whatever sand and silt enters the canal settles in this reach. As it is very expensive to remove all the spoils and high banks in the 2 or 3 miles of the canal, attempt was made to see if by any other method the entry of silt into the canal could be reduced.

Last year experiments were designed to study the effect of varying the openings of different undersluice and head regulator gates at the Anderson Weir at Rhondia as a means of reducing entry of silt in the head reaches of the Damodar Canal. For this purpose, observations of the silt intensity in the river and the canal were taken first with uniform openings and then by increasing the undersluice gate openings from the left to the right hand bays and raising the canal regulator gates more and more from the upstream end.

The suspended silt content in the river water was observed last year at a point just upstream of the weir. This was the most suitable point for observation. But the river silt content as observed at this point was almost invariably less than the canal silt content. This was probably due to the flow being obstructed ahead of the point of sampling so that the coarser particles settled down.

(3) River Research Institute, West Bengal, Annual Report, 1947, pages 100-104.

This year observations of the river silt content were simultaneously made at a point downstream of the river. It was found that the upstream river silt content was less than the canal silt content but the silt content of the river at the downstream was slightly higher than in the canal. Hence this downstream river silt was considered to be more representative of the river and was taken into account.

This year observations were taken keeping head regulator gates uniformly open all throughout but varying the undersluice gates in three ways : (1) by operating the gates uniformly, (2) by increasing the gate openings from the left to the right, and (3) by increasing the gate openings from the right to the left.

As before the ratio between the silt intensity in the canal water and the river water has been studied. It was found that the ratio is almost always less than unity.

The ratios as obtained for different observations were grouped into three classes according to the nature of the openings of the undesrsluice gates. The mean value of the ratio for each class and the standard deviation from this mean are given in Table 10 C. 1.

TABLE 10 C-1

Nature of opening of under-sluice gate	Number of observation	Mean ratio	Standard deviation	Coefficient of variation
1. Uniform	62	0·777	0·223	28·6
2. Increasing from left to right	27	0·668	0·170	25·4
3. Increasing from right to left	28	0·882	0·111	12·6

It is found from the above that the ratio of the canal silt content to the river silt content decreases on the average when the gates are operated increasingly from the left to right, by about 14 per cent., but increases by nearly the same amount when the relative position of the gates is reversed.

The significance of the difference observed in the mean values and standard deviations has been tested by Fisher's t and Fisher's Z respectively which are

$$t=\frac{\bar{X}_1-\bar{X}_2}{\sqrt{\dfrac{(n_1-1)s_1^2+(n_2-1)s_2^2}{n_1+n_2-2}}}\times\frac{1}{\sqrt{\dfrac{1}{n_1}+\dfrac{1}{n_2}}},\text{(degrees of freedom}=n_1+n_2-2)$$

$$\text{and } Z=\tfrac{1}{2}\log_e\frac{s_1^2}{s_2^2},\text{ (degrees of freedom}=n_1-e_1\text{ and }n_2-1)$$

where $\bar{x}_1$ and $\bar{x}_2$ are the means, and s_1 and s_2 are the standard deviations of the two samples whose difference is tested and n_1 and n_2 are the number of observations for each. The values of t and Z and their significance are given below :—

Difference between samples	t	Significance of t	z	Significance of z
1 and 2	.. 2·263	Significant on 5 per cent.	0·270	Not significant.
1 and 3	.. 1·042	Not significant ..	0·696	Highly significant.

Thus the decrease in the ratio of the canal silt by river silt in the case of increasing gate openings from the left to right is significant. In the other case the difference between s_1 (uniform opening) and s_2 (for increasing gate openings from right to left) is highly significant and as such the t-test is not applicable. However, this latter test, when applied, does not pronounce the increase in the ratio to be significant.

It appeared that by working the undersluice gates in such way as to give more opening from left to right, the silt entry into the canal is reduced. It was, therefore, suggested that this method be followed at Rhondia Headworks during the next working season.

During 1947 monsoon season the canal headregulator gates were opened uniformly. It is proposed to try these experiments during 1948 by varying the canal regulator gate openings from the uniform opening as practised last year.

(4) COMPARISON OF SAND-EXCLUSION AT THE NOSE OF THE OUTER BANK AND AT THE RAISED SILL OF THE APPROACH CHANNEL UNDER PRESENT CONDITIONS AND WITH OUTER BANK EXTENDED TO POSITION Q—EXPERIMENTS IN $\frac{1}{80} : \frac{1}{40}$ PART-WIDTH, SUKKUR BARRAGE MODEL [4]

The following experiments were carried out to test the effect of the extended bank on sand-exclusion :—

Experiment (1) :—

Exclusion at the nose and at the raised sill with the existing unextended Outer Bank.

[4] Central Waterways, Irrigation and Navigation Research Station Poona, Annual Report Technical, 1947, pages 177-178.

The $\frac{1}{80}$: $\frac{1}{40}$ part-width model was laid in Koregaon sand of 0·34 m. m. diameter to the 1947 soundings.

The following discharges and gauges were maintained:

Q River	275,000 cusecs
Q Approach Channel	45,000 cusecs
Q Tail Channel	27,000 cusecs
Q Pocket	18,000 cusecs
Outfall gauge	R. L. 196·8
Water level downstream of Barrage	R. L. 193·3

Surface and Red lines of flow were observed by wool thread in the Approach Channel.

Experiment (2) :—

The Outer bank of the Approach Channel was extended to position Q and the scour-hole at the present nose was filled up to R.L. 175 as obtained in previous experiments. Same gauges and discharges as in Experiment 1 were maintained.

Experiment (3) :—

Though actually no appreciable scour occurred on the Approach Channel side of the nose in Experiment 2 the bed was laid down to R. L. 160 *to impose the worst conditions*, assuming a scour-hole would form, though of less depth than under existing conditions due to the flat slope of the extend nose.

RESULTS

For efficient sand exclusion at the raised sill, the flow in the Approach Channel should follow the right bank; actually under the present conditions it hugs the Outer Bank. This partly neutralises the curvature of the Approach Channel intended to exclude bed material at the raised sill. The exclusion at the nose has also deteriorated since the bed material is attracted into the deeper scour-hole on the Approach Channel side.

With continuous extension of the Outer Bank to the position Q, the flow conditions were appreciably improved, both at the entrance of the Approach Channel and at the raised sill.

There was no return flow at the entrance of the Approach Channel and the surface flow followed the Right Bank, thus establishing the desired favourable curvature for sand exclusion.

Inspite of the wider entrance of the Approach Channel when extended to position *Q*, the width of bed material drawn by the Approach Channel was less, being only 450 feet against 530 feet under present conditions. Exclusion at the raised sill was also noticeably better in Experiment 2, as the distance of the divide from Right Bank was only 110 feet as against 160 feet under present conditions.

With scour-hole laid down to R. L. 160 on the Approach Channel side of the nose in Experiment 3—a *severe* assumed condition—slightly more bed material was attracted into the Approach Channel as compared to Experiment 2 ; but this was still better than with the present unextended nose. Exclusion as the raised sill remained as good as in Experiment 2.

CONCLUSIONS

With the extension of the Outer Bank, as already recommended, the deep scour-hole near the nose in the Approach Channel will accrete and slips in the Right Bank will be prevented.

Sand Exclusion at the nose and at the raised sill will be materially improved and will remain so even if a scour-hole develops at the entrance of the Approach Channel.

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(5) EFFECT OF CHANGE IN REGULATION OPERATIONS OF THE RIGHT POCKET ON SAND EXCLUSION—EXPERIMENTS IN $\frac{1}{80}$: $\frac{1}{40}$ PART-WIDTH SUKKUR BARRAGE MODELS[5]

Since the construction of the Approach Channel, it has been usual to carry out the annual *scouring operations* of the Right Pocket towards the end of *Abkalani*, in the middle of September. The Pocket thus flushed remains clear till the next floods. With river discharge above 3 *lakhs* cusecs, there is excessive silt in suspension and the pocket gets silted fairly quickly, especially in the reach up to 900 feet upstream of the barrage. A stable channel section sufficient to carry the designed discharge, however, remains and there is no progressive reduction in area of the pocket as is generally supposed. Nor can the quantity of the deposited silt, when scoured later, make a significant addition to the silt charge entering the canals.

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(5) Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report, 1947, pages 179-185.

The September scouring operation is carried out *with canals closed* and a maximum discharge of 27,000 cusecs is run in the Pocket and 18,000 cusecs in the Tail Channel, thus making a total of 45,000 cusecs in the Approach Channel. In previous years, the canals were kept closed during *abkalani* scouring in June, July or August; but this naturally seriously interfered with irrigation operations.

Experiments were carried out to compare sand exclusion at the raised sill of the offtake into the Right Pocket under existing regulation operations and under change in regulation.

It was found that increase in the pocket discharge to 24,000 cusecs maintaining ratio $Q_{T.C.}/Q$ pocket$=1\cdot5$ *deteriorated* the exclusion at the raised sill. Reversal of this ratio apparently reverses the favourable curvature at the raised sill. This is only to be expected. So, either an increase in pocket discharge maintaining the discharge ratio of 1·5 or altering this ratio even for a short period is likely to draw more sand into the pocket.

Thus, with the change in scouring operations, there would be a higher sand charge drawn by the pocket and, consequently, the N. W. Canal and the Rice Canal would draw more sand than at present. The Dadu Canal is likely to be the worst affected compared to existing conditions.

By admitting more water in the Pocket, as compared with the discharge in the Tail Channel, in order to scour the Pocket (while keeping the canals running), the favourable curvature at the raised sill is reversed and large quantities of sand will be drawn by the Pocket, which would subsequently find their way into all these canals.

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(6) SUPPLY OF SAND-FREE WATER TO JAMSHEDPUR PUMP-HOUSE [6]

INTRODUCTION

Both the Jamshedpur Steel Works and the town were previously served by water pumped from the Subarnarekha River. Recently, the Dimna *nala* has been dammed some six miles north east of Jamshedpur and water supply for the town is tapped from this Dimna reservoir; but water for the Steel Works continues to be pumped from the river just upstream of the pick-up weir. The existing pumping installations have a total capacity of 31,850 gallons per minute. The existing suction main is a four feet diameter, cast iron pipe with its open end located midstream, some distance above the weir, at a point where bed water heavily charged with bed sand is evidently sucked in without hindrance.

The pumps can be worked continuously all the year round except for the danger of the heavy charge of sand being drawn during the flood season, which causes heavy wear and tear of pump casings and impellors, thus reducing their life and efficiency. Hence, during floods, the pumps are being worked at

(6) Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report Technical, 1947, pages 187-190.

present to meet only the minimum requirements. In spite of this precaution, in the monsoon about 5,000 cubic feet of sand collects daily in the main common suction chamber inside the Pump-house and has to be promptly removed ; and, as a result, the pump casings have been severely pitted and have slowly disintegrated by attrition.

LOCATION OF THE PUMP-HOUSE

The first requirement, was to design a practically sand-excluding intake.

Various methods have been tried and tested, in the past, for sand exclusion mainly in connection with the design of canal headworks. None has been found as satisfactory, in general, as locating the intake on the *concave* (*i.e.*, outer) bank of a curve, where the bed water containing sand is naturally deflected away and only sand-free, top water enters the intake works.

The *first* essential was, therefore, to locate the intake near the downstream end of the available concave bend, though well upstream of the cross-over of the deep-water river channel, so as to obtain the maximum benefit of the favourable river curvature. A suitable point near the existing Pump-house was suggested for the intake, on these considerations.

Two counteracting features, reducing the efficiency of sand exclusion must, however, be borne in mind :—

(1) The effect of curvature is diminished in periods of low river by ponding of water caused by the weir ; and

(2) In high floods, in spite of repositioning of the intake, the bed sand thrown into suspension by turbulence and intermixing of the water at different depths may still be sucked in, to a certain extent, unless arrangements are made to tap only the top filaments and to insure minimum agitation.

In addition to correctly relocating the intake, some additional arrangement to prevent the entry of sand in the suction pipe was, therefore, considered necessary.

INTAKE TOWER

An intake tower, well streamlined in plan to minimise turbulence and with an arrangement for tapping *surface* water at all river stages, as shown was, recommended.

PIPE LINES FROM INTAKE TOWER TO PUMP-HOUSE

Alignment.—The best alignment can only be determined from a contoured plan, or else from cross-sections and L. section, which were not available

Size.—It is always desirable to maintain as high a water level in the Tower as possible in keeping with the cut-off necessary to pass the required discharge into the chambers over the movable gates. It is, however, preferable that pipe diameters should be designed for the *lowest* water level inside the Tower. The size of the pipe being thus determined, further reduction of pipe diameter

Fig. 10.C.3. (1918)

merely to increase the scouring velocity—to keep the pipe flushed—is not permissible as the capacity of the pipe will then be inadequate at low water level in the chamber. Moreover, for flushings, high velocities can be generated by proper control of the Tower gates and gate valves in the intake pipes. These considerations led to the adoption of four feet spun-concrete pipes as leading mains from the twin inlet chambers and five feet diameter pipe as common main connecting the four feet pipes with the Pump-house sump.

Regulation.—In low river, all the gates excepting the bottom gates are to be closed so that water will *only* enter through the peripheral vents, flow down the intake pipes and collect in the middle chamber, from where it is carried to the Pump-house by the mains.

With the oncome of floods, the Water level will first sufficiently rise to permit taking water over the top of the *lowest* gate. The intake pipes are then to be closed with the covers and the outer gates fully lowered.

Depth of water for passing 133·55 cusecs over the pair of gates is estimated to be about one foot.

The gates are *always* to be so operated, as far as possible, that only about one foot layer of surface water is tapped.

The inner chambers of the Tower can easily be closed—one at a time or both together—by lowering all the gates and draining the chambers by opening the sluice valves in the water mains. During this time—which will not be frequent or long—water for works can be supplied from the second chamber of the Tower, if one is closed down ; or, if both chambers are simultaneously closed, from the cooling pond or else from the Dimna reservoir.

Constructional difficulties.—As water supply can be taken from the Dimna reservoir during the period of construction, no serious difficulty is anticipated in constructing the proposed Tower.

As soon as the water level has gone sufficiently low, the existing diversion weir below Pump-house can be cut to drain off the water. The foundations can then be built in the dry without having recourse to Coffer-dams or a ring *bund*.

After the work has sufficiently progressed and is carried above foundations the breach in the weir can be made good.

(7) MODEL OF NANGAL CANAL EXCLUDER (7)

A complete model of the Nangal Barrage was examined for current direction, movement of bed material, and ramp formation against the canal—regulator, as a pre-requisite to the design of the excluder.

A study of the current directions in a discharge equivalent to 10,000 cusecs when the entire supply was going to the canal, showed that flow took place in the right half of the river. In the left half there was back water.

It could be concluded from this study that in order to have the desired curvature of flow in the Pocket, the gates on the right should be given greater openings, stepping down to the left. Adopting the above system of regulation, discharges equivalent to 20,000 and 50,000 cusecs were examined. A photograph of the model is given in Figure 10 C. 3.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. GUPTA in introducing item (1) said that the new supply channel taking off from Bhimgoda weir drew in huge quantities of shingle owing to the favourable curvature of flow, as a result of this a large sum of money was spent every year in shoal clearance works. The possibilities of a shingle excluder were studied in a vertically exaggerated part model of the weir and the head regulator to scale 1/40 horizontal and 1/24 vertical.

A three tunnel shingle excluder (tunnel size 15 feet × 5 feet each) provided in bay No. 1 of the under sluices proved very effective in preventing the shoaling of the new supply channel.

DR. BOSE then introduced items (2) and (3). About item (2) he had nothing to say except that the experiments proved that the silt entry into the Damodar canal would be reduced by manipulating the gates.

The other experiment done was that of a sandy model and the exclusion of silt was required. They found that the entry of silt into the canal in the model was different depending on the condition of the model.

As soon as water was let into the canal so that the canal got wet, the sand got picked up. So also when there was a shower of rainfall before the running

(7) East Punjab Irrigation Research Institute, Amritsar, Annual Report, 1947, page 43.

of the canal, the amount of silt into the canal was found less. It might be due to the kind of sand at the place. Here the sand was micaceous. This was not noted in the earlier stages. He liked to know the experience of other officers in similar experiments.

Items (4) to (6) were then introduced by RAO BAHADUR JOGLEKAR. Speaking about item (4) he said that under existing conditions, at the entrance of the approach channel, there was a scour-hole 75 feet deep at its maximum, with high velocity flow in the middle of the approach channel and return flow along the nose and the right bank. Berms which formed due to the slack return flow along the right bank "slipped" into the deep scour-hole, in 1945 and 1946, and the Bunder wall and Sukkur road were consequently damaged.

To prevent this, last year's model experiments showed that the extension of the outer bank by about 500 feet ending in a nose with a 5 : 1 slope on the approach channel side would be a satisfactory solution.

It was expected at the time that in addition to the prevention of slips the extended outer bank would improve sand exclusion at the nose and at the raised sill in the approach channel.

Under existing conditions, due to the deep scour-hole, the flow did not follow the right bank as it should for optimum sand-exclusion at the raised sill. With flared entrance and extended nose with flatter slope, there was no appreciable scour and favourable curvature of flow as desired was produced.

Exclusion at the nose was materially improved. On the whole exclusion at the raised sill was much better than with the existing nose.

Thus the extension of the outer bank as recommended would not only prevent slips in the right bank but was expected substantially to improve exclusion at the nose and at the raised sill.

Introducing item (5) he said that since the construction of the approach channel, the usual practice was to carry out the annual scouring operation of the right pocket in the middle of September with a maximum pocket discharge of 27,000 cusecs and a tail channel discharge of 18,000 cusecs, and canals closed in the middle of September. The pocket thus flushed remained clear up to the next floods when due to the excess silt charge it got silted up very quickly. Even then a stable channel sufficient in waterway to carry the pocket discharge was maintained; there was no progressive reduction of area of the pocket and original bed levels were restored after the scouring operations.

The barrage authorities, however, desired additional scouring (operations of short durations in June, July and August, with the implied object of having a reserve for silt deposition on the bed of the pocket which, in their opinion, would be otherwise carried into the canals.

In the model observations of lines of flow and distribution of seeds (dropped upstream of the raised sill) between the tail channel and pocket showed that the favourable curvature for sand exclusion at the raised sill was completely destroyed and most of the bed material entered the pocket.

So they concluded that (*i*) either an increase in the pocket discharge or the reversal of pocket to tail channel discharge ratio would lead to deterioration in sand-exclusion from the right pocket and, (*ii*) scouring operations with canals open were undesirable.

Referring to item 6 he said that two alternative designs, differing mainly in arrangement for tapping water at low river level, were suggested.

Dr. Uppal introduced item (7) and said that they carried out experiments to obtain minimum silt entry into the Nangal power channel. The idea was to obtain the minimum silt entry upstream of the regulator and the alignment of regulator with respect to the barrage. Different curvatures which existed on the old headworks were tried. The one which existed at Rupar was giving the best results and that was adopted. They found that concentration of silt was moved to the centre of the barrage. They tried several methods and had been successful to a certain extent. Further work was in progress.

Rai Bahadur C. L. Handa observed that regarding silt exclusion, that this had been a matter that had drawn the attention of a number of engineers; one such was the question of shingle problem on the Nangal Barrage. There the ponding up was considerable. The pond level had risen 25 to 28 feet above bed level. Similarly the main headache at Madhopur headworks was about shingle exclusion. The Nangal canal being a hydro-electric canal, one could ill-afford to have any shingle in it. Dr. Uppal's suggestion for the exclusion of silt should be very important in this connection.

Mr. Ghotankar said that since this was not a problem of excess silt entering at the head, controlling sand entry by regulation would affect the regime of the canal downstream. He enquired whether this had been considered.

Regarding the silt trouble in the Damodar Canal Mr. Ghotankar said that he understood it was colloidal silt; if it was not and if it was micaceous and floated, then one could very well allow water from the downstream end, and pond it up. That was what they tried in their experiments.

In reply to Mr. Ghotankar Dr. Bose said that the samples were collected from the downstream of the undersluice gates and as such, could be expected to give reliable samples.

Rao Bahadur Joglekar referring to item (3) said that it was stated in the Bengal Report that silt observations were done downstream of the river and it was found that the silt content downstream was slightly higher than the canal and hence the downstream river silt was considered to be more representative of the river.

Was the increase in silt content downstream due to increased turbulence? If so, how could it be represented?

As stated last year in the Bengal Report the main trouble of silting in the canal was due to slipping of high banks and not due to excess entry of sand at the head.

Dr. Bose enquired whether silt did not deposit in the proposed water tower at Jamshedpur Power-house.

Rao Bahadur Joglekar replied that the water tower was proposed to be located on the outside of a curve where the river had always been deep and hence there was no danger of bed sand being drawn into the tower. Besides, there was an arrangement of gates by which only surface water would be admitted into the tower and even if some silt deposited in the water tower arrangements had already been proposed for scouring away the silt.

It was decided to keep the subject on the agenda.

DISCUSSION BY THE BOARD

The Secretary said that seven items were discussed at the Research Committee meeting (page 951). There was no resolution. Draft on the proposed C.B.I. Publication on the subject was still awaited from Dr. Uppal.

Rai Bahadur C. L. Handa observed that when planning hydel channels the problem of shingle excluders should receive an importance which could have been ignored when only irrigation was the service to be rendered by these channels. Firm power, once having been promised to the public, it is imperative that carrier channels should function without any interruption even during the highest floods. The present practice was to close the canals at head when the flood exceeded a certain limit and opportunity was taken to flush the pockets. This facility would cease to exist. In the case of Nangal Canal which would have a lake of about 30,000 acre feet at the headworks with a depth of the pond something like 40 feet the trash at the head regulator of the canal had been kept at 25 feet. He wanted to submit that the problem of shingle excluders should be understood in all its implications. Silt excluders would be one solution. After that one must be prepared to put in the canal one shingle extractor which would have certain advantages. The turbines and generations would be attacked by silt unless fullest precautions were taken. Hence the expenditure in power canals would be much more than in the irrigation canals. Again neither the silt excluder nor the shingle extractor could solve the problem 100%. One must, therefore, be conscious of the limitations of these devices and be prepared to do something more. For instance in the case of Nangal, head regulators with all the above precautions one could not give up the prospect of clearance of huge amount of conglomeration. Four months intensive experimental work had been done at Madhopur to find out satisfactory solution of the shingle problem. He showed three photographs which represented one of the latest attempts which Dr. Uppal had worked out. They hoped to provide maximum shingle exclusion by this method.

THE SECRETARY then requested East Punjab Officers to expedite action regarding the Central Board of Irrigation Publication on "Silt excluders and ejectors."

RAI BAHADUR C. L. HANDA said that their Chief Engineers were thinking of supplementing their staff and he promised to do the best.

DR. H. L. UPPAL said that with one or two extractors at proper places it was expected that all the silt which was harmful might be extracted.

DR. N. K. BOSE wanted to know from the electrical engineers as to what size of the particle their machinery would tolerate. There was no use excluding all silt. Certain amount of silt should go with the water.

MR. S. A. GADKARY said that the smaller the size allowed, the better. Even 0·2 mm. would be fairly big. Only that suspended could be allowed. Even ordinary sand had deteriorating effect on hydraulic machinery.

THE CHAIRMAN (MR. S. A. GADKARY) continuing said that the size of sand or silt allowable depended largely on the head and velocity. If the head and velocities were small even larger particles did not do much harm. Generally all kinds of sand were to be excluded.

RAI BAHADUR C. L. HANDA said that in the case of large dams the problem did not arise. The low and medium head projects were the real worries.

MR. R. L. NARAYANAN said that it also depended on the abrasive property of the sand. Abrasive sands were to be excluded.

RAI BAHADUR C. L. HANDA remarked that sand was usually abrasive.

MR. S. N. GUPTA brought out a problem which confronted them in U. P. Ganga Canal at mile 44 above Nirgajni Power-house was trained by three spurs on its right bank for equitable distribution of silt leading to the twin turbines. A vane was also constructed last year to improve vortex formation in the left machine with a view to restore the original load which had slightly reduced. It was known after sometime that the blades of the left turbine were worn out more as compared to previous years. This was attributed to the accelerated churning action of more silt particles now coming into the left bay owing to the putting in of the vane. Was this enhanced action on blades due to this ?

THE CHAIRMAN (MR. S. A. GADKARY) observed that on the face of it the turbine was not capable of taking the load it had been designed for. It was not anything due to silt but only a mechanical phenomenon.

MR. R. L. NARAYANAN said that it must be due to uneven flow of water to the two turbines that they did not synchronise.

11C. Outlets or Sluices

PRELIMINARY NOTE

This subject was introduced for the first time on the agenda of the Research Committee in 1947. The subject has been dealt with comprehensively in the Punjab Engineering Congress Paper No. 264, by Messrs. Mahbub and Gulhati. It is understood that the authors propose to re-publish this work in book form and after making considerable additions and modifications to their Punjab Engineering Congress Paper. There was no contribution or discussion at the 1947 meeting of the Research Committee under this head.

THE YEAR'S WORK

DISCUSSION BY THE RESEARCH COMMITTEE

There was no contribution on the subject and no discussion was held.

DISCUSSION BY THE BOARD

THE SECRETARY said that there was no contribution and no discussion at the Research Committee Meeting.

12C. Other Works

PRELIMINARY NOTE

This subject head was introduced in 1947 to facilitate the discussion of the design of irrigation works other than those which are included on the agenda as separate items.

The following items were discussed at the 1947 Research Committee Meeting :—

(1) Fluming the Tungabhadra Low Level Canal between mile 27/4 and 28/4.

(2) Reduction of afflux at sluices, culverts and inverted siphons.

(3) Siphoning of the Cumbum tank sluices.

Recent Literature.

(1) Bhatia K. M.—Design of cisterns downstream of regulators—Central Board of Irrigation Journal, Vol. 4, No. 2, April 1947.

(2) Khangar S. D.—Design of siphon on the Thal Canal in the Punjab—Central Board of Irrigation Journal Vol. 3, No. 3, July 1946.

HE YEAR'S WORK

The following items were discussed at the 1948 Research Committee Meeting :—

(1) Experiments to test the suitability of the position of the lock span proposed in the Sind design.

(2) Proposal for a combined road and rail bridge over the river Ganga at Patna or Mokameh Ghat.

(3) The Hagari Aqueduct for the Tungabhadra Canal.

(1) EXPERIMENTS TO TEST THE SUITABILITY OF THE POSITION OF THE LOCK SPAN PROPOSED IN THE SIND DESIGN(1)

It is contemplated in the design of the Lower Sind Barrage to site the lock span just *outside* the left pocket divide wall with a Fender Lock wall 210 feet upstream of the weir ; the length being fixed from the consideration of the biggest craft that is likely to use the lock. Length of the biggest craft is assumed to be 200 feet and an extra 10 feet is provided in the length of the approach or Fender Lock wall.

(1) Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report Technical, 1947, pages 191-195.

(2) Experiments were carried out in the 1/250 : 1/50 vertically-exaggerated model with the proposed position and arrangement of the lock :—

(*a*) To test the suitability of the position proposed in the design ; or otherwise to determine the best location of the lock span from the point of view of affording a safe approach ; and

(*b*) To determine the optimum length of the approach wall from the point of view of least suction caused by the adjacent river spans, drawing the craft towards them during floods.

(3) Experiments revealed the *unsuitability* of the proposed position from the following points of view :—

(*a*) Due to curved, and persistently turbulent flow, associated with the diving flow at the nose of the left pocket divide wall, this position left much to be desired in respect of satisfactory and safe approach conditions.

(*b*) Even if there were perfect conditions of parallel flow, the velocity of 10 feet per second to 12 feet per second (or sometimes even more) in this zone would certainly make the berthing of a vessel along the approach quay to the lock in this position a difficult task, and one of considerable risk both to the vessel as well as to the quay.

(*c*) Though the Fender Lock wall produced the desired effect of creating slacker water within the area bounded by it and the left pocket divide wall, a ship model showed that the manoeuvring of the vessel would still be difficult, even after it crossed the first hurdle at the nose of the left pocket divide wall, in view of the secondary turbulent region formed just outside the nose of the Fender wall.

(*d*) A number of tests were also carried out with this Fender wall varying in length from 150 feet to 400 feet ; but neither a short nor a long wall could reduce the hazards of turbulence.

(4) At this stage, the following alternative positions for the lock were considered :—

As there is only one canal on the right bank the lock could be constructed in it with two approach channels, one to the upstream and the other to the downstream side of the barrage, with the following possible advantages :—

(*a*) Economically, the cost of the approaches might be more than offset against the cost of the Fender wall and much lower costs of lock gates, lock bridge, *etc.* ;

(*b*) Hydraulically, it would avoid the hazards of turbulent and diving flow and afford easy and safe approach conditions for vessels ; it would also largely be unaffected by the operation of the Barrage gates

The following disadvantages, however, more than counterweighed the advantages :—

(*i*) this arrangement would interfere with the operation of the canal based on considerations of irrigation demand ; the excessive heading-up, caused in the head reach above the first cross regulator, at periods when the canal would be running with a low capacity factor, would lead to periodic silting in this reach ;

(*ii*) and it is also likely that the right arm channel may build up its bed and may not provide sufficient draft at the entrace and/or exit of the approach channels linking the lock to the river.

The second proposal was to site the lock at the extreme right end of the left arm barrage to take advantage of the Central Island as an approach quay for the vessels.

But this position was also ruled out after model tests for the following reasons :—

(*i*) it would have made the approach conditions even more difficult than if located just outside the pocket, due to greater turbulence and higher velocity along the shank of the island ;

(*ii*) there would also be less draft available on the downstream side of the lock as it lies on the convex, and so the *sanding*, side of the curve in the left arm channel ; and

(*iii*) it would have been hazardous to manoeuvre the crafts along the submerged slopes of the left arm shank of the Central Island.

The third and *best* alternative site considered was just *inside* the left pocket having the following merits ;

(*i*) as the discharge in the pocket is limited, the maximum velocity is never likely to exceed five feet per second (and under ' still pond ' up to river discharge≡452,000 cusecs, only 2·82 feet per second) with the regulation phases recommended and so would never be subjected to such turbulence as would be objectionable to the safe manoeuvring of ships ;

(*ii*) while working the ship model, it was observed that there is no draw of current *away* from the divide wall in its vicinity, thereby producing unfavourable conditions for berthing a vessel alongside of it ; besides there is likely to be a deep water channel from the left guide bank, upstream of the pocket, to the divide wall nose ;

(*iii*) it would also enable handling a Large volume of traffic at a time, due to sufficient space being available inside the pocket ;

(*iv*) the twist and scour at the nose of the Fender lock wall could also partly aid exclusion of sand rolling along the narrow strip besides the pocket divide wall.

All doubts whether the requisite draft will still be available in case the pocket gets badly silted along the divide wall, as in the case of the left pocket of the Sukkur Barrage, were disproved by rigorous model tests made for this purpose. These tests showed that, under the worst conditions, sufficient depth would still be maintained to give the required draft and scouring operations could quickly and efficiently increase depths, if and when necessary.

An appreciable reduction in silting of the left pocket of the Sukkur Barrage, since it has been run under "semi-open" flow conditions, for discharges higher than 350,000 cusecs, is convincing of the superiority of the "semi-open" flow recommended; if similar regulation is enforced, there will be no excessive silting in the left pocket at the Kotri Barrage.

Moreover, the three left bank canals at Sukkur, nearest to the barrage line, (*i.e.*, Khairpur Feeder East, Khairpur Feeder West and the Eastern Nara Canal), drawing a maximum aggregate discharge of 16,000 cusecs, are accommodated in a width of 750 feet along the Head Regulator face line, as a result of which the mean velocity in the zone of the left pocket contributing this discharge to these three canals is hardly more than 1·25 feet per second against three feet per second in the case of the Pinyari Feeder of the Kotri Barrage drawing 20,600 cusecs and accommodated in a restricted width of hardly 314 feet. Thus, the higher velocity obtainable in the Kotri Barrage design will always maintain a minimum depth of 11 feet (against three feet in the Sukkur Barrage) below the pond level, based on $\frac{V}{V_0}$ considerations in these zones.

One series of experiments were aimed at working out the discharge that can be boosted in the pocket and pushed through the lock approach channel to keep it clear of silt, with the optimum permissible opening of the lock span gate, without vitiating the sand exclusion from the pocket or affecting the regulation phases.

After deciding upon the best position of the lock, experiments were carried out for determining the optimum length of the approach or Fender wall with the following main considerations in view;

(*a*) to avoid danger to the crafts by the suction caused by adjacent spans; and

(*b*) to provide satisfactory approach conditions by ensuring sufficient draft at the entrance to and in the Approach.

As the pocket gates adjacent to the lock span will not be kept open to the same extent as the barrage gates, suction effect was hardly observed to extend 100 feet upstream of the weir line and thus did not influence the length of the approach wall.

For consideration (*b*) above, four different lengths = 337 feet, 275 feet, 230 feet and 210 feet upstream of the weir, were tested in the model. After running rising and falling cycles with the optimum permissible openings of the

lock span and with silting in the dead water region of the left pocket artificially reproduced right up to the pond level (*i.e.*, the *worst* hypothesis)—it was observed that the longer wall was better than the shorter wall for the purpose of clearing the silt from, and in front of, the lock Approach Channel Table 12 C. 1.

TABLE 12 C. 1

Length of the Fender wall (feet)	Minimum R. L. down to which sand was scoured in front of the wall
337	R. L. 50·0
275	R. L. 53·0
230	R. L. 56·0
210	R. L. 60·0

Had it not been for the diving mid-depth flow at the nose of the pocket divide wall extending 150 feet below it—under the worst conditions of the regulation—the longest lock wall would have been, therefore, preferred. With the lock wall 337 feet in length, the noses of the two walls will be 293 feet apart and crafts would have steady water for a length of hardly 143 feet (obtained by deducting 150 feet from 293 feet); which is less than the length of the biggest craft (200 feet) likely to use the lock. For satisfactory approach conditions there must remain a distance of at least 200 feet—preferably 250 feet—essential to avoid vessels being directly drawn to, and impinging against the nose of the lock wall. To meet this requirement, the Approach wall would have to be not more than 230 feet long.

LENGTH OF DOWNSTREAM LOCK WALL

In the Project design, the downstream lock wall is proposed to be 290 feet in length measured from the weir crest, the length being fixed by the maximum length of the craft and the minimum additional distance required for operating the flap gates of the lock.

Model experiments showed that this length was suitable from the point of view of approach and exit to the lock as the nose of the downstream divide wall of the left pocket, *i.e.* the point of the maximum turbulence, is sufficiently away (by about 155 feet) from the nose of the downstream lock wall.

The maximum draft of 10 feet required for the craft is also likely to be insured by the regulation of the lock span.

Hence, the length of the downstream lock wall as contemplated in the design may be retained.

RECOMMENDATIONS

(1) The lock should be constructed in the left pocket, *i.e.*, immediately to the left of the left pocket divide wall.

(2) The length of the upstream Fender Lock wall should be 230 feet.

(3) The left pocket divide wall should be used as the approach to enter the lock with bollards fitted to it at intervals of 100 feet from the upstream end of the lock.

(4) Through the lock span and spans adjacent to it supplies available owing to the difference between the actual and capacity discharge drawn by the canals should be utilized to minimize sanding of the lock span.

(5) The left pocket gates may be kept three feet higher than the other barrage gates (instead of 2 feet previously recommended) to enable flushing of of the lock span.

(6) The length of the downstream lock wall may be kept 290 feet as contemplated in the design.

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(2) PROPOSAL FOR A COMBINED ROAD AND RAIL BRIDGE OVER THE RIVER GANGA AT PATNA OR MOKAMEH GHAT (2)

INTRODUCTION

The relevant portions of the 1947 Report by Mr. Fenton on the proposal for the construction of a combined road and rail bridge over the Ganga river were forwarded to the Director for his opinion. The results of Mr. Fenton's investigations showed that of the two alternative sites—Patna and Mokameh—the Patna site, at the present time and in the near future, is unfavourable from an engineering point of view. On the other hand, it was stated that the Government of Bihar is extremely keen on the proposed bridge being located at Patna and it is probable that even if the cost of the bridge at Patna were appreciably greater than one at Mokameh, the Patna site would be the one decided upon.

STATION RECOMMENDATIONS

These were made, as desired, by way of replies to the queries raised by Mr. Fenton, the Author of the Report, as mentioned below.

Question 1 (*a*) **Do you agree with the conclusions in the Report that a bridge at Patna at the present time or in the immediate future is not a feasible engineering undertaking ?**

(b) **Is it your view that by conducting model experiments vital information, not apparent in the course of ordinary investigations may be obtained and a clearer picture of the worthiness or otherwise of the site produced ?**

These two questions were answered simultaneously.

(2) Central Waterways, Irrigation and Navigation Research Station, Poona, Annual Report, Technical, 1947, pages 203—218.

It was mentioned that though, *prima facie*, Mokameh site is better, the existing river conditions at Patna with suitable training measures do not summarily preclude the practicability of a bridge in this vicinity.

Observations in the model in respect of velocities, flow lines, velocity and discharge distribution, formation of shoals, scour and tendency of the river to stabilise or wander, are useful for the design of guide *bunds* and for deciding the relative merits and demerits of various sites.

If the bridge must be constructed at Patna, it was considered that the alternative alignment half a mile upstream of the project alignment—subject to verification on a model—appeared *superior* from various considerations elucidated in the Note.

Question 2 (a) **Do you agree that the site suggested for a bridge at Mokameh is one offering the security with which the Report credits it ?**

(b) **Do you consider that any model experiments are necessary to support or modify the conclusions of the Reports in this respect ?**

Except that the south bank is apparently not so 'worthy' as at Patna, it was agreed that the site suggested for a bridge at Mokameh offers reasonable security.

Though it is desirable that the construction of the bridge should be expedited, it was considered that the model investigations *must precede* the construction as they will provide invaluable indications useful in the design of bridge details.

Question 3. **If your reply to Question 1(a) above is to the effect that in your view the Patna site is not to be ruled out of court but is one which offers the possibility of a reasonably secure birdge, how will you compare its worthiness and security with those of the suggested bridge at the Mokameh site ?**

Certain objections to the Patna site, as detailed in the 'Note' were admitted. On the other hand, the Mokameh site was accepted to possess equally obvious merits emphasized in the 'Report'.

Though Mokameh site was thus accepted as secure, it was made clear that it was impossible to be unequivocal and reject Patna site without model experiments. Even if Mokameh site is exclusively selected, it was emphasized that model experiments would give greater confidence in the choice made.

Question 4. **Do you agree with those matters relating to scour, depth of foundations, design of training *bunds etc.*, dealt within the report, in respect of both the alternative sites—Patna and Mokameh, or do you consider that they need modifications in any respect ?**

(i) **Site :**

As mentioned above, it was stated that model experiments were desirable to determine the precise local position of the alignment at Mokameh or Patna.

In the case of Mokameh site, the Project alignment appeared acceptable; omission of artificial protection to the south bank (as suggested in the Report) was also acceptable unless experiments or subsequent experience warrant its necessity.

(ii) **Peak Discharges :**

It was pointed out that extrapolation beyond the limited range of measured data is undependable. In the Report extrapolation was made much beyond the discharges actually observed.

Secondly, the coefficient of 0·85 used in the Report, for coverting the surface velocity to mean velocity along the vertical is not a fixed constant, especially in curved flow, and is generally of the order of 0·89.

These errors being *counteracting*, the peak discharges were *provisionally* accepted pending further measurements.

(iii) Waterway :

In the Report, the waterway was calculated according to the Lacey formula $P_{10}=2\cdot667\ Q^{\frac{1}{2}}$, and in arriving at the number of spans an amount equal to twice the width of the wells was deducted from the distance between centres of piers to obtain the effective waterway per span as is recommended by Lacey. An allowance of about 10% over Lacey width was thus provided when the effective waterway was worked out. This waterway thus appeared to be adequate when considered together with the divergence of shape from Lacey.

(iv) Scour and depth of piers, scour at shanks and heads of guide *bunds* :

Maximum probable depths of scour at the heads and shanks of guide *bund* were calculated in addition to scour depths at piers worked out in the Report

(v) **Grip of Pier foundations** :

With maximum scour=$2R$ (Lacey) and a grip length=R, the depths of foundations obtained were practically the same as assumed in the Report (based on Gale's grip) and hence were recommended for adoption.

It was further suggested that the depth of pier foundation for each pier should be worked out separately for the value of 'f' appropriate to the pier.

It was also agreed that no pitching is necessary round the piers provided sufficient grip is insured.

(vi) **Training works** :

It was stated that it is extremely difficult and costly to protect guide *bunds* with falling aprons when the sand bank contains interspersed layers of clay; in such cases, it is better to construct a new sand bank and pitch it with stone.

For determining the lengths of guide banks, it was opined that model experiments would be necessary.

Radius of the nose of the guide banks adopted in the Report was, it was observed, smaller than recommended by Gale and hence full apron protection was recommended.

(vii) **Piers** :

It was suggested that cut-and ease-waters of the semi-circular type should extend sufficiently above the high flood level.

(viii) **Provision against earthquake** :

Being in an area of considerable seismic activity, bridge piers were recommended to be designed against horizontal forces due to acceleration of at least $g/5$ in combination with longitudinal forces resulting from breaking and traction.

Question 5. **Have you any other general criticisms or suggestions to offer in respect of any of the above issues or any other that may occur to you ?**

One or two points such as post-bridge action of the river on the south bank, advisability of considering river training measures other than guide banks *etc*. were suggested.

In the final report Mr. Fenton has offered remarks on the above points raised by the Director. With a view to present a comprehensive picture, Mr. Fenton's remarks and the Director's rejoinders thereto are given in the ' Note '.

Further examination of the data as well as remarks by Mr. Lacey will be undertaken at the time of the model experiments.

(3) THE HAGARI AQUEDUCT FOR THE TUNGABHADRA CANAL (3)

Studies made during 1946 on a model 1/400×1/100 of the river Hagari with the proposed aqueduct to study the scours round piers, *etc.*, were described in the annual report for that year. As stated therein further experiments were continued on the model after introducing certain modifications resulting from changes in design.

MODEL

The model was exactly the same as described in the previous report except in regard to the details of the aqueduct: this was now made of 56 spans of 35 feet each. Experimental operations were also similar.

EXPERIMENTS

The main experiments made on the previous model were repeated and the concentration of scour at the left abutment was observed to be persisting. Qualitative tests made by shifting the abutments or by providing additional water way through the left side embankment did not prove encouraging. Finally, therefore the flood bank proposed by the Executive Engineer, Hagari Division, was introduced and its effect tested. It was found to be effective and the addition of a training *bund* downstream indicated further improvement. A study of these scour contours resulting from a one hour run with each of these two designs in comparison with the contours obtained in the previous experiments showed that concentration of flow and consequent scours at left abutment could very much be reduced by the arrangement now proposed.

CONCLUSION

A low flood bank along the left margin and connecting the left abutment to high ground about four furlongs higher up will prevent lateral flow and eddying at the abutment which would cause deep scours at the abutment. An extension of this flood bank by about 400 feet downstream in the form of a training *bund* will further improve the low pattern.

DISCUSSION BY THE RESEARCH COMMITTEE

MR. C. V. GOLE introduced item (1) and said that experiments for testing the practicability of the position of the Lock-span proposed by Sind, just outside the left pocket divide wall, were carried out in the 1/250 : 1/50 V.E. model

(3) Irrigation Research Station, Madras, Annual Report, 1947, page 10.

with the length of the Fender wall=210 feet. It was observed that Lock span located outside the divide wall could not provide workable conditions required for safe approach by vessels owing to the curved, diving, persistently turbulent flow at the noses of the left pocket divide wall and the Fender wall. He explained the different positions tried in experiments.

RAO BAHADUR D. V. JOGLEKAR introduced item (2).

MR. T. P. KUTTIAMMU introduced item (3).

DR. N. K. BOSE referring to item (1) enquired whether there would not be turbulence at the nose of the divide wall.

Mr. C. V. GOLE replied that the pocket was regulated under still-pond conditions for discharges upto 300,000 cusecs. For discharges above 300,000 cusecs there would be restricted semi-open flow. The velocity in the pocket would not be more than five feet per second and the ships could easily approach along the left guide bank into the pocket and then enter the lock without any trouble.

It was decided that the subject should remain on the agenda.

DISCUSSION BY THE BOARD

THE SECRETARY said that three items were discussed at the Research Committee meeting (page 970). There was no resolution.

13C. Staunching of Canals and Embankments

PRELIMINARY NOTE

In 1945 it was decided to issue a Board publication on the subject "Staunching of Canals"*.

In 1946 meeting the Board accepted the recommendations of the Research Committee and changed the title of the subject from "Staunching of Canals" to "Seepage Losses from canals and their prevention" with the following sub-heads :—

(*i*) Seepage Losses from canals

(*ii*) Staunching of canals

(*iii*) Economics of lining

In 1947, the subject 'Seepage Losses from canals and their prevention' was divided into two parts. Staunching of canals and Embankments and Seepage Losses. The latter is now included under Section A—Hydrology, item 5, 'Regeneration and Losses'.

Under 'Staunching of canals and embankments' two sub-heads were approved,

(*i*) Materials (*ii*) Economics.

(i) MATERIALS

PRELIMINARY NOTE

At its 1947 Annual meeting held in December 1947, the Board passed the following resolution :—

> "In view of the necessity of providing protection to the water face of earthen dams and embankments against wave wash, the Board resolved that experiments be undertaken on field scale with regard to the use of various materials for the purpose including (*a*) graded and treated soils, (*b*) *Surkhi* concrete and (*c*) Cement concrete.
>
> Resolved further that investigations be undertaken also to evolve suitable methods for protection against wave wash."

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*Please see Preliminary Note on "Regeneration and Losses" item 5, Section A—Hydrology.

The following items were discussed at the 1947 Research Committee Meeting :—

(1) Staunching of canals (the use of *usar* soil).

(2) Experiments on staunching of canals with *surkhi* mortar.

Recent Literature.

(1) Young W. R.—Low cost linings for irrigation canals—Indian Concrete Journal, Vol. 21, No. 10, October 1947.

(2) Lauritzen C. W. and Israelsen O. W.—Wests Canal lining studies—Western Construction News, Vol. 22, No. 5, May 1947.

(3) Elfman, S., Civil Engineer, The State Power Board, Sweden.—Stopping seepage in the gravel esker at Namforsen—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 19.

THE YEAR'S WORK

The following items were discussed at the 1948 Research Committee Meeting: –

(1) Tests on materials for staunching of canals and embankments

(2) Staunching of the Godavari left bank canal.

(3) Experiments continued with staunching of canals with *surkhi* mortar.

(1) TESTS ON MATERIALS FOR STAUNCHING OF CANALS AND EMBANKMENTS (1)

ABSTRACT

The various types of materials tested for their suitability for canal lining are given below :—

(*i*) Bitumen or Tar-Impregnated fabrics.

(*ii*) Shell soil stabilizer.

(*iii*) Soil cement mix.

(*iv*) Soil sand cement mix.

The nature of tests performed on a small scale in the laboratory under rigidly controlled conditions are :—

(*a*) determination of seepage losses through the lining under different conditions of temperature and head of water, (*b*) deterioration under wet and dry conditions, (*c*) coefficient of expansion with respect to temperature and moisture.

A material not yielding satisfactory results in any one of the above mentioned tests was considered unsuitable for lining purposes. If, however, it stands all such tests in the laboratory, it is to be retested on a larger scale under field conditions.

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(1) East Punjab Irrigation Research Institute, Amritsar, Annual Report 1947, pages 44-56.

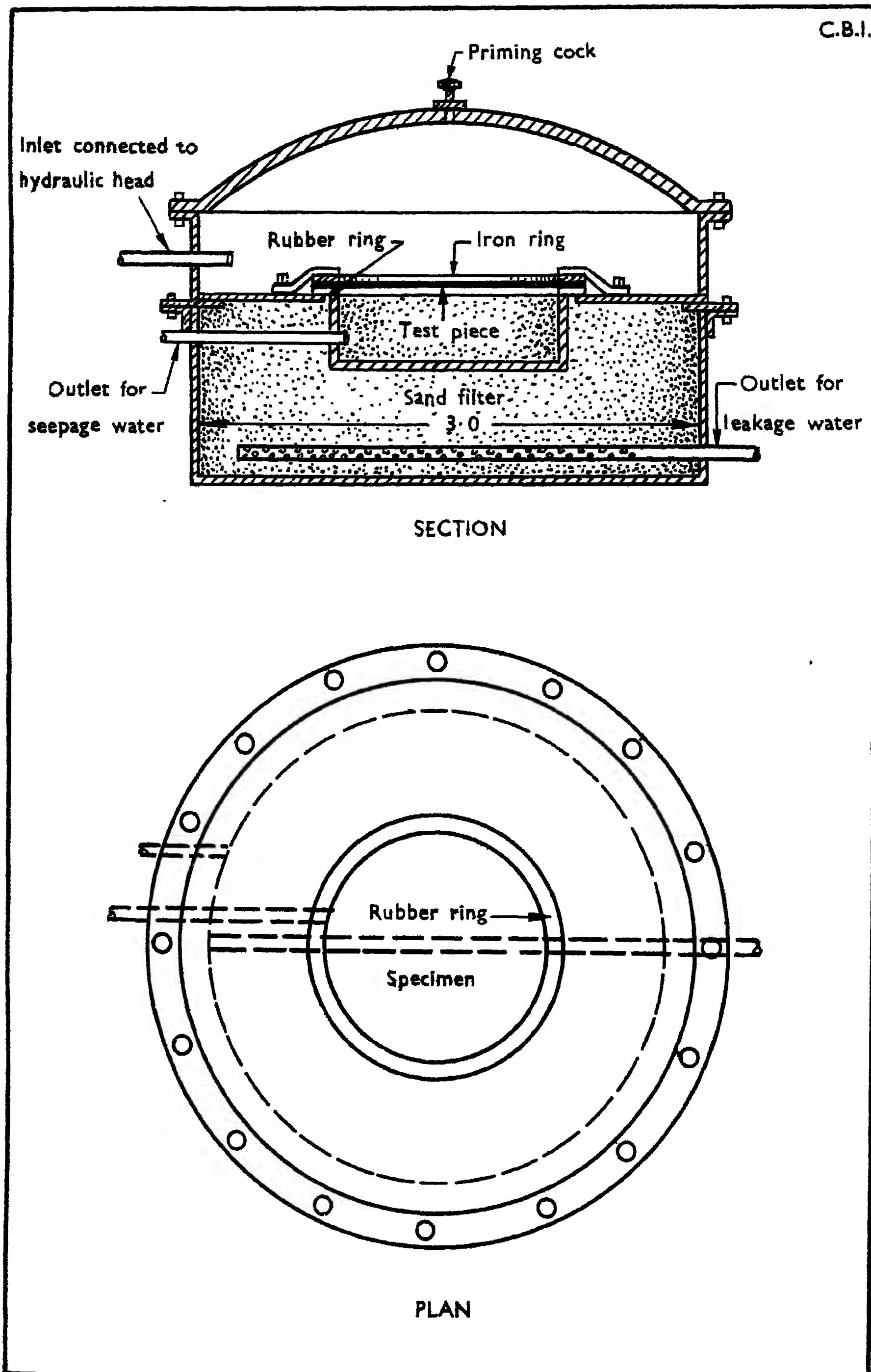

Figure 13C.1 :- *Showing permeability apparatus for bitumen impregnated fabrics*

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BITUMEN IMPREGNATED FABRICS

It is generally a jute fabric impregnated with relatively soft low-softening-point bitumen and coated with hard high-softening-point bitumen on both sides. It was originally designed for surfacing runways and taxiways. Such fabrics are usually called P.B. S., which is the abbreviation for Prefabricated Bitumen Surfacing. They were supplied by Burmah Shell, Bitumen Department, Calcutta, for testing them for their suitability as a canal lining in its present form.

The tar impregnated fabric is a similar product, the impregnating material being Tar instead. It was supplied by ' Shalimar Tar Products Co., Ltd.'

SEEPAGE TESTS

The details of the experimental technique and procedure are given in the Annual Report of the Punjab Irrigation Research Institute Lahore for the year ending April 1943. After some experience it was, however, realised that the apparatus employed for this purpose suffered from the following two drawbacks :—

(1) Although at the outset the test piece was invariably fitted leaktight, yet it was often observed that it did not remain as such when subjected to 10 feet or higher heads of water. The water leaking through the jointing space would get mixed up with the water seeping through the body of the fabric. The estimation of seepage losses was thereby rendered inaccurate.

(2) The sand column, on which the test piece was resting, was usually kept in an unsaturated condition. As the seepage water had to percolate through it before it could be collected and measured a portion of it would get absorbed by the sand particles. The results thus obtained could scarcely be relied upon especially when the seepage losses were low.

The first source of error was eliminated by equipping it with a device by which leakage water, if any, is not allowed to mix up with seepage water. It consists of a cylindrical vessel having its diameter slightly less than that of the opening of the tank and fitted with a side tube for collection of seepage water. Coarse sand is packed in this vessel as well as in the space enclosed between it and the tank. Thus water derived from two different sources could be collected separately.

The second defect could be removed by keeping the outlet point meant for collection of seepage water at a level flush with the surface of the sand column. The entire sand column could thus be kept saturated and the seepage water could no longer get absorbed by the sand particles. The diagrammatic sketch of the improved device is given in Figure 13 C.4.

Two samples cf Burmah Shell and eleven of Shalimar have been tested fo seepage losses in addition to those described in the previous report. At leas two pieces of each specimen were tried and the average value of losses obtained The results are given in Table 13 C. 1.

TABLE 13 C. 1

Serial No.	Number of sample	Name of supplying company	Head of water in feet	Seepage in cusecs per 106 square feet.
1	M1	Burmah Shell ..	15	1·2
2	M2	Bitumen Department	15	1·5
3	410*a*	..	20	No seepage
4	410*b*	..	20	Do.
5	411*a*	Shalimar ..	20	Do.
6	411*b*	Tar	20	Do.
7	TM/CAN	Products ..	20	Do.
8	TM/HESS	Limited	10	Excessive seepage.
9	TM/TWL	..	10	Seepage.
10	2/TM/CAN/D ..	..	20	No seepage.
11	2/TM/CAN/S ..	..	10	High seepage.
12	TM/CAN/D	..	9	20·2.
13	TM/TWLL	..	9	Excessive seepage.

Out of the lot of 13 samples, only six which have stood the seepage test were subjected to other tests.

DETERIORATION TESTS

The deterioration of the fabrics was examined in two ways, (*a*) by keeping the test pieces immersed in water for about six months or so and retesting for seepage losses similarly and (*b*) by keeping them in the open, fully exposed to the atmospheric conditions, for months on, and studying the condition of the impregnated material, regarding (*i*) softening at high temperature, (*ii*) cracking at low temperature, and (*iii*) disintegration with lapse of time. As it also rained twice during the observation period, the fabrics were also subjected to alternate conditions of wetting and drying.

(*a*) Only five samples, *viz.*, 410*a*, 410*b*, 411*a* and 411*b* and TM/CAN which stood the initial seepage test were further examined fcr a similar test after six months' immersion period. Again no seepage was found to occur through them even at a 20 feet head of water at 32° C kept for a fortnight.

(b) When kept in the open, all of them were found to soften at about 52° C to 57° C. Sweating at the under surface was particularly noticeable. No cracking occurred at low temperatures in any case. After the expiry of four months no signs of disintegration were perceptible, though the colour of the fabric changed in every case.

EFFECT OF MOISTURE ON THE DIMENSIONS OF THE FABRIC

The description of the apparatus and experimental procedure adopted for this problem is given in the annual report of the Punjab Irrigation Research Institute Lahore for 1945 (page 37, Fig. 65).

The experimental results are given in Table 13 C. 2.

TABLE 13 C. 2.

Serial No.	No. of the sample	Name of supplying company	% linear variation	Nature of variation
1	410a	..	0·42	Contraction.
2	410b	Shalimar ..	0·40	Do.
3	411a	Tar	0·42	Do.
4	411b	Products	0·37	Do.
5	TM/CAN	Limited	0·39	Do.
6	Canvas unimpregnated	..	3·5	Do.

From the results it is clear that though the tar impregnation has reduced the co-efficient of expansion to about one tenth of that of the untreated fabric, still it is considerable as compared with that of other linings.

Again while conducting experiments in winter it was found to suffer from another drawback. As the fabrics were supplied to us in rolls, so naturally they would get curved at many places along the length. A test piece had to be kept under pressure for a suitable period in order to straighten it. On dipping it in water it would slowly develop that curvature again. This difficulty was encountered only in winter, not in summer. This is a serious drawback, as it would not be easy to manipulate such a material in the field, especially in the winter season.

SOIL—SHELL SOIL STABILIZER MIX

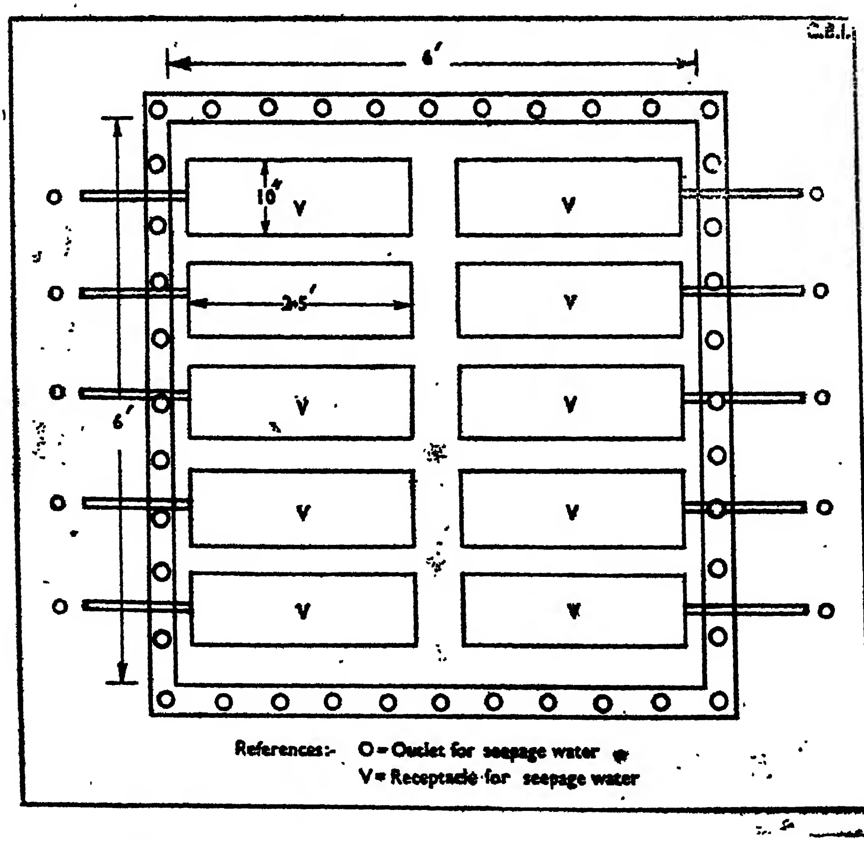
A material, called Shell soil stabilizer, was supplied by Burmah Shell, Bitumen Department, Co. It has been utilised for consolidating bases of runways during the war. It is a viscous substance obtained by dissolving a bituminous material in some oily solvent.

A method for preparing the mix had to be developed. After some preliminary attemps it was found that the material did not mix well with a dry soil. The quantity of water, which should be added to the soil, for obtaining uniform distribution of the stabilizer, turned out to be about 20% of the soil by weight

Preparation of the mix.—The soil to be used was air dried, coarsely powdered sieved through a one cm mesh-width sieve and placed in a mixing tray. The requisite amount of moisture was added and thoroughly mixed. Finally the required amount of the stabilizer was added and mixed up. The mixture was allowed to dry up, as it could not be compacted to optimum density at this moisture content. As the optimum moisture was attained, a portion of the stuff was spread uniformly over the inverted filter contained in the lining tank to a thickness of two inches. Thus the lining block was constructed to the requir-ed thickness and allowed to cure properly before subjecting it to any head of water.

SEEPAGE TESTS

The details of the experimental procedure employed for such tests have already been given in the Punjab Irrigation Research Institute, Lahore, Report 1943, page 29, Fig. II. 3. The experimental technique used before, being de-fective in certain respects, had, however, to be improved upon. The sketch of the improved device is given in Figure 13 C. 2.



Figures 13 C. 2 : *Showing improved plan of seepage tank.*

The results of experimentation are given in table 13 C. 3.

TABLE 13 C. 3

Serial No.	Composition of the mix, by weight. Soil : Stabilizer.	Cross-sectional dimensions of the block	Thickness of the block in inches	Optimum D. B. density (C. G. S. units).	Head of water in feet	Seepage per million sq. feet (in cuscecs)
1	100 : 7½	36 inches diameter	6	1·65	20	0·4
2	100 : 7½	36 inches x 13 inches	4	1·64	15	0·8
3	100 5	Do.	4	1·63	15	1·1
4	100 : 5	Do.	3	1·63	15	1·4

After about a month a sudden increase in the seepage losses was observed, indicating thereby the setting in of the deterioration of the lining. On opening the tank the lining was found to have softened and growth of moss was perceptible in certain patches. These tests indicated that this type, though more costly, was decidedly inferior to the soil-cement mix.

SOIL CEMENT MIX

Preparation of the Mix.—After some preliminary attempts the following procedure was adopted. The soil was dug, air-dried, coarsely powdered and sieved through 1 cm. mesh sieve. The optimum moisture content of the soil cement mixture of varying composition was determined. The materials composing the mix, *viz.*, soil and cement were taken, measured for correct proportioning, placed in a mixing tray and mixed to a uniform colour by turning with square-pointed shovels. The requisite amount of water *viz.*, optimum moisture content was added and turned by shovels so as to get a fairly uniform moisture content throughout the mass of the mix. The mixture thus obtained was spread uniformly over the inverted filter to a thickness of about two inches and compacted by means of rammers to optimum density. Thus the lining slabs were constructed to the required thickness and allowed to cure properly for about four weeks, when they were subjected to a hydraulic head, which was increased in steps gradually from 1 foot to 20 feet.

SEEPAGE TESTS

In case of lining types, not investigated till then, it was difficult to foresee their behaviour with respect to seepage losses *etc.*, and it was considered expedient, keeping in view saving of both time and money, to prform the pereliminary tests on a smaller scale. An attempt to devise an apparatus for this pose met with success. The sketch is given in Figure 13 C. 3.

It consists of three main parts :—

(1) A cast iron cylinder AB is divided into two compartments and provided with a flange at the top. The side tubes S and L communicate with the inner and outer compartments respectively.

(2) The cylinder CD is provided with flanges at both ends. It is fitted on to AB by means of a system of nuts and bolts with a rubber padding in between the flanges to make it leak-tight.

(3) The lid P has a cast-in flange and a central hole to be closed by a rubber cork containing two openings, one for insertion of a thermometer and the other for communicating with a head of water.

EXPERIMENTAL PROCEDURE

Both the inner and outer compartments of AB are packed with coarse sand, (the grade of sand in the outer one being one somewhat coarser) to a level nearly half an inch below the top edges of AB. The test specimen is constructed *in situ* in the cylinder CD on the sand base provided in AB. As the seepage tests are also to be conducted at higher hydraulic heads, say 20 feet or more, it is quite necessary that the cylinders and the lid be fitted up perfectly leak-tight.

The water leaking through the space lying along the surface of contact of the cylinder and the test sample is collected in the outer compartment of AB and escaped from the side tube L, while water seeping through the lining gathers in the inner compartment. In this way the error arising from the mixing up of the two kinds of water is eliminated. Having cured the lining block for about four weeks the experimental vessel is filled with water, closed, leak-tight and subjected to a hydraulic head varying from one inch to 20 feet in steps. It is kept at a particular pressure for at least one week to study the effect of time factor on the seepage losses. The results for soil-cement mixes varying in composition, thickness, and dry bulk density examined for initial seepage losses, are given in Tables 13 C. 4 to 13 C. 7.

TABLE 13 C-4

1st set—The clay content of the soil used was about 13 per cent.

Serial No.	Composition of the mixture		Thickness of the block in inches	Optimum M. content	Optimum D.B.D.	Head of water in feet	Seepage in cusecs per 10^6 square feet
	Soil	Cement					
1	100	16	6	16%	1·79	20	0·00
2	100	12	4	16%	1·78	20	0·00
3	100	10	4	16%	1·78	20	0·00
4	100	7½	4	15%	1·77	20	0·00
5	100	5	4	15%	1·78	20	0·00
6	100	5	3	15%	1·75	20	0·00
7	100	5	2	15%	1·78	20	0·00

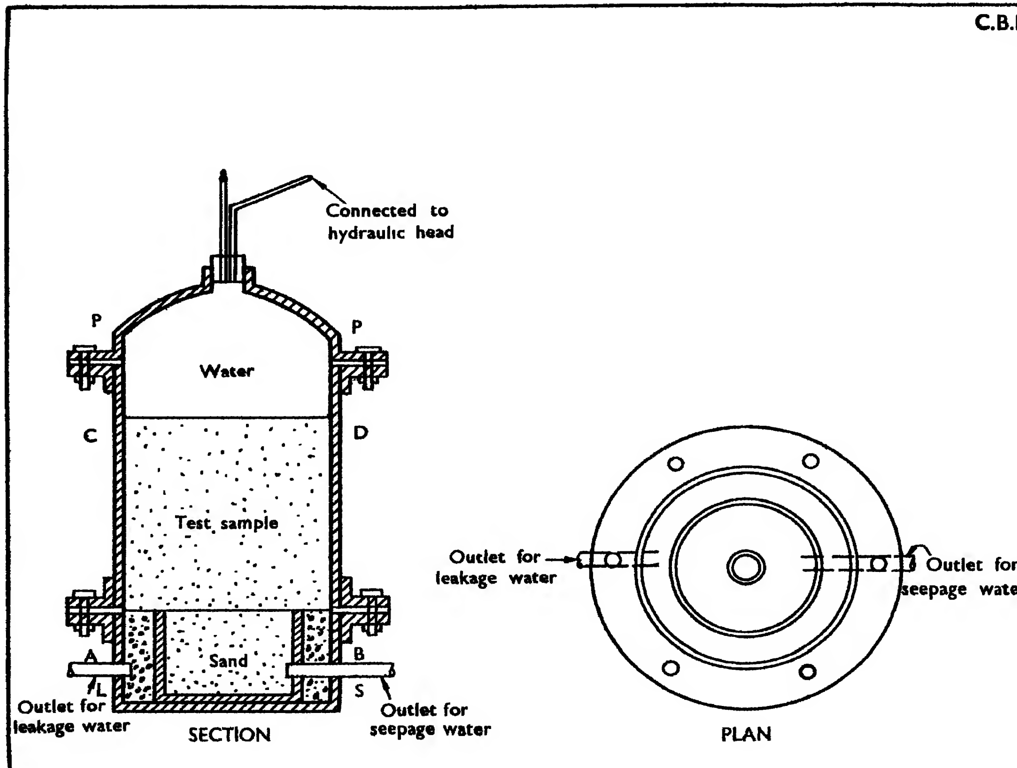


Figure 13C.3:- *Showing permeability testing apparatus*

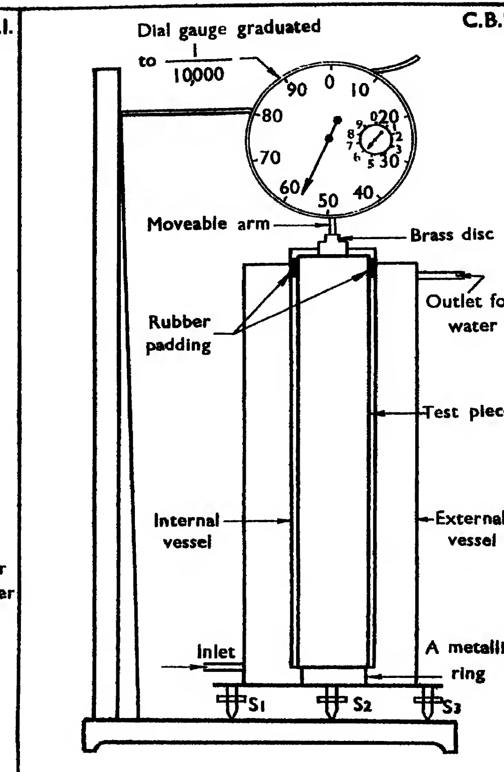


Figure 13C.4:- *Showing coefficient of expansion apparatus*

Reg. No. 2737 XDD (C) '50-1,210 (P.Z.O.).

TABLE 13 C. 5

2nd set—The clay content of the soil used was about 16 per cent.

Serial No.	Composition of the mixture		Thickness of the block in inches	Optimum M. content	Optimum D.B.D.	Head of water in feet	Seepage in cusecs per 10^6 square feet
	Soil	Cement					
1	100	5	4	15%	1·77	20	0·00
2	100	5	4	20%	1·56	20	0·00
				(m. c.	optimum)		
3	100	5	3	15%	1·76	20	0·00
4	100	5	2	15%	1·77	20	0·00

TABLE 13 C. 6

3rd set—The clay content of the soil was about 25 per cent. It contained a high percentage of humus.

Serial No.	Composition of the mixture		Thickness of the block in inches	Optimum M. content	Optimum D.B.D.	Head of water in feet	Seepage in cusecs per 10^6 square feet
	Soil	Cement					
1	100	5	4	21%	1·69	20	0·06
2	100	5	3	21%	1·68	20	0·12
3	100	5	2	21%	1·68	20	0·21

TABLE 13 C. 7

4th set—This was an alkaline soil having clay and salt content about 20·5 per cent and 0·3 respectively.

Serial No.	Composition of the mixture		Thickness of the block in inches	Optimum M. content	Optimum D.B.D.	Head of water in feet	Seepage in cusecs per 10^6 square feet
	Soil	Cement					
1	100	5	4	17%	1·78	20	0·00
2	100	5	4	21%	1·57	20	0·00
				(m. c.	optimum)		
3	100	5	3	17%	1·77	20	0·00
4	100	5	2	17%	1·76	20	0·00

DETERIORATION TESTS

This aspect was studied in three ways : (*a*) The seepage test described above was prolonged for several months and the increase in seepage, if any, noted. (*b*) The tank was opened after completing the initial seepage test and the blocks allowed to dry up for about a month and retested for seepage losses similarly. (*c*) Constructing cylindrical blocks, 6 inches × 6 inches, of each type. Curing them properly and immersing them in water contained in a metallic reservoir, which was kept upstairs fully exposed to the weathering agencies. The hardness and weight of the blocks were observed regularly. In another set of experiments the blocks were subjected to alternate conditions of wetting and drying and similar observations taken.

The results of deterioration tests are given below :—

(*a*) Regarding prolonged seepage tests of blocks kept immersed, continuously for about three months, no increase in the seepage losses occurred in any case except humus soil—cement mix.

(*b*) After letting the blocks dry up in the winter season no cracks appeared anywhere at the surface. On retesting for seepage losses no signs of deterioration were perceptible. In summer, however, cracks appeared and, when retested, the blocks, 2 inches in thickness, gave high seepage losses, showing thereby that the cracks had either penetrated the entire thickness or, at any rate, a sufficient depth, so as to mar the water proofing properties of the lining. It may be mentioned here that no space was left between the adjacent blocks to allow for their expansion caused by variations in temperature or moisture. Their behaviour when provided with adequate expansion joints remained to be studied.

EXPANSION AND CONTRACTION OF SOIL CEMENT MIX

The appearance of cracks in the lining during summer led to the study of its co-efficient of expansion with respect to moisture and temperature. .. Only the effect of moisture has been studied so far.

A block, 77 cm. × 10 cm. of 100 : 5 : : Soil : Cement mix was constructed and cured properly. It was placed in a metallic vessel and kept in a vertical position. Water was filled in the vessel to a level flush with the upper edge of the block. The change in the length of the block was measured by means of a dial gauge reading to $\frac{1}{10,000}$ of an inch. A diagramatic sketch of the apparatus is given in Figure 13C. 4.

It consists of a double walled cylindrical vessel, about 6 inches in diameter, 78 cm. high, placed on a plane iron plate fitted with levelling screws. The lining block, 77 cm. × 10 cm. constructed and cured in the usual way, is placed in the inner vessel. The sideways displacements of the blocks are checked by

inserting rubber paddings between the blocks and the vessel. Its vertical displacements caused by variations in moisture and temperature are, however, not kindered in any way. The external vessel contains water and is meant for reducing the error due to fluctuations in the room temperature. The lining block is adjusted in a vertical position with the help of the levelling screws, S_1, S_2, S_3. A brass disc, 5 cm. in diameter and 1 cm. thick, fits in a cylindrical groove made at the central portion of the top surface of the block. At the centre of this disc is drilled a hole about 5 mm. in diameter and 3 mm. deep. The arm of the dial gauge is so adjusted as to make its lower end touch the bottom of this hole. As the block expands or contracts, the arm of the dial gauge moves up or down displacing thereby the needle of the gauge. The position of the needle, when it assumes a stationery state, is noted.

For studying the effect of moisture on the linear dimension, water is added in the vessel to a height flush with the upper edge of the block and the position of the needle observed at suitable time intervals. From the length of the block under test and the maximum displacement of the needle the required co-efficient of expansion can be computed. The temperature of water should be kept constant during the experimentation period. If it varies, the error involved should be duly accounted for. For replica results the arm of the gauge as well as the block should be kept in a vertical position. In order to test the reversibility of the phenomenon water is taken out of the vessel and the block allowed to dry up. The reading of the needle, when it undergoes no further displacement, is noted.

The effect of moisture on expansion and contraction of the blocks made of soil alone was also studied. The experimental results are given in Table 13 C. 8

TABLE 13 C. 8

Serial No	Composition of the mix		Clay content of the soil used	Change in length	Remarks
	Soil	Cement			
1	100	5	13	0·162	Expansion on moistening
2	100	5	13	0·164	Contraction on drying
3	100	5	16	0·167	Expansion on moistening
4	100	5	16	0·167	Contraction on drying
5	100	0	13	3·4	The expansion coef. of soil blocks was studied in a different way
6	100	0	16	3·5	

An inspection of the results indicates that (1) the co-efficient of expansion (with reference to moisture) of the soil cement mix is reduced to one twentieth of that of soil alone (2) the phenomenon is reversible, as on drying up the block resumes its original length. This fact shows that the cracks, which appear in the soil cement lining on drying up, would most probably get closed up on remoistening, provided the cracks do not get filled up, partly or completely, with any sand-like rigid stuff.

DETERIORARION TESTS

For examining this aspect, cylindrical blocks, 6 inches × 6 inches, were constructed out of a given mix as well as of the soil used therein. Some of them were kept immersed in water continuously for several months, while others were subjected to alternate conditions of wetting and drying. The shape and hardness of the blocks was studied in the case of the first set ; while for the 2nd set, in addition to the above characteristics the weight of the blocks was also noted. Table 13 C. 9 contains the experimental results.

TABLE 13 C. 9

1st set.

Serial No	Composition of the block		Remarks
	Soil	Cement	
1	100	0	Deterioration set in as soon as dipped in water ; disintegrated completely within an hour and a half ; neither retained its shape nor hardness.
2	100	1	Retained its shape for one day ; disintegrated partly after a few days. The sides stood at an angle greater than the angle of repose say about 70°
3	100	2	Retained its shape but softened slightly.
4	100	3	Showed very slight charge in shape or hardness, after several months.
5	100	4	The blocks having 4 and 5 % of the cement content showed no change in shape and hardness even after six months.
6	100	5	

In case of the 2nd set blocks composed of only 100 : 5 : : soil : cement were tried. No change in their shape or hardness was perceptible even after the expiry of some months. A light decrease in their weights did occur after three months. It is not deemed advisable to deduce any conclusion from these experiments at this stage. This work will have to be continued in order to make an exhaustive study of each factor.

SOIL AND CEMENT MIXTURE

In an attempt to reduce the co-efficient of expansion of the soil cement mixture, sand was used as a third constituent in addition to the two components of the cement mixture. The procedure adopted for the preparation *etc.*, of the mixture was similar to that employed for the soil cement mixture. The results of seepage tests are given in Tables 13 C. 10 and 13 C. 11.

TABLE 13 C. 10

Soil No. 1.

The clay content of the soil was 16 percent. and the mean particle size of sand = 0·28 mm. diameter.

Sr. No.	Composition of the mix			Thickness of the block in inches	%optimum moisture content	Optimum D.B.D.	Head of water in feet	Seepage in cusecs per million sq. feet.
	Soil	Cement	Sand					
1	80	5	20	4	15	1·77	20	0·00
2	66·6	5	33·3	4	15	1·75	20	0·00
3	33·3	5	66·6	4	15	1·76	20	0·75

TABLE 13 C.11

Soil No. 2

Clay content of the soil=20·5 percent. Mean diameter of the sand=0·28 mm. Salt content of the soil=0·3 percent

It was an alkaline soil.

No.	Composition of the mix			Thickness of the block in inches	% Optimum moisture content	Optimum D.B.D.	Head of water in feet.	Seepage in cusecs per million sq.ft.
	Soil	Cement	Sand					
1	80	20	5	4	15	1·78	20	0·00
2	66·6	33·3	5	4	15	1·79	20	0·00
3	33·3	66·6	5	4	15	1·78	20	0·50

DETERIORATION TESTS

The seepage tests performed on blocks kept immersed in water for about two months did not show any increase in the seepage loss. Only the blocks which gave no seepage initially were retested. No other test had been performed with this lining type.

COEFFICIENT OF EXPANSION WITH REFERENCE TO MOISTURE

The experimental technique and procedure is similar to that employed for the soil cement mixture. The results are given in Table 13 C.12.

TABLE 13 C.12

Sr. No.	Composition of the mixture			Dimensions of the block	% change in length	Nature of variation
	Soil	Sand	Cement			
1	80	20	5	32×15×10 cm.	0·12	Expansion on moistening
2	80	20	5	do	0·13	Contraction on drying
3	66·6	33·3	5	do	0·081	Expansion on moistening
4	66·6	33·3	5	do	0·081	Contraction on drying
5	100	0	5	do	0·164	Expansion on moistening
6	0	100	10	do	0·006	do

For making a quantitative estimate of the reduction in the coefficient of expansion, similar blocks composed of soil-cement and sand-cement were also tried for this test. As the block made out of sand : cement : : 100 : 5 crumbled off on handling due to lack of adequate cohesive strength, another containing 10% of cement was examined.

From the results it is obvious that sand is effective in reducing the coefficient of expansion with respect to moisture. The effect of moisture on sand—cement block is comparatively much less. The cracks should, therefore, appear in soil—sand—cement blocks to a lesser extent than in those containing soil—cement only. This type of lining compared with that of soil cement possesses greater abrasive but lesser cohesive strength.

(2) STAUNCHING OF THE GODAVARI LEFT BANK CANAL (²)

The Godavari Left Bank Canal cuts through *murrum* from mile 8/7 to 10/5. Water seeping through this cutting of the canal led to the rise of the sub-soil water table year after year. There was a large area of damaged land —adjacent to the canal and it was feared that if no preventive measures were taken more land would go out of cultivation. There were constant and numerour complaints from villagers about serious damage to their lands. It was, therefore, proposed to line the canal in this section with sodium clay. An estimate was framed for the same in 1942 in which 5½ tons of sodium Carbonate were provided for sodiumising 400,000 cubic feet of soil required for the purpose.

It was, however, found by the Executive Engineer, Nasik Irrigation Division, that owing to war conditions the cost of $Na_2\,CO_3$ had risen enormously and was prohibitive. He also believed that the water levels in the damaged areas had gone down due to (*i*) rush rotation system of irrigation adopted since; and (*ii*) the working of wells in the area for irrigation. There were of late no complaints also from the villagers—pressing for the reclamation of lands.

It was also suggested by the Executive Engineer, Nasik Irrigation Division, whether the salts naturally occurring in the area could be utilised for sodiumising the soil instead of commercial $Na_2\,CO_3$.

Accordingly the area downstream of the canal near Rui village was resurveyed and it was found that the damage by salt and water logging remained unchanged and that sub-soil water levels in wells were as high as before.

There was no salt incrustation at the surface enough for collection on a large scale for use in the preparation of sodium clay. Only slight salt was present in the locality. Samples of naturally sodiumised soils were, therefore, taken from two sites for testing.

One was in S. No. 45 of Rui which was slightly salt affected; (it contained 0·477% of salts) another was in S. No. 101 which was salt affected and contained 1·25% of salt.

At each site the soil was heaped (5 feet × 5 feet×1½ feet) on—*tarvad* branches to facilitate drainage. A basin 6 inches deep was made at the top of each heap. A depth of 3 inches of water was maintained at the surface of each heap by watering daily.

The process lasted for two months. More water was required in the beginning but later on the quantity required was fairly constant about one cubic foot or 0·48 inch per square feet, and samples were collected from each heap – before, and after leaching.

(²) Poona Irrigation and Research Division, Poona, Public Works Department. Bombay, Annual Report 1947, pages 26—31.

The process of leaching reduced the salt content to the same level in both heaps (from S. No. 45 and S. No. 101); leaching was probably unnecessary in the former cases (S. No. 45) as shown by the comparative salt and clay content before and after leaching.

Conductivity was reduced from 750 to 500 in the case of heap in S. No. 45 and from 3,500 to 600 in the heap from S. No. 101. Clay content has increased in the latter case as a result of leaching. There was not much change in *pH* values.

Before treatment, the soil in S. No. 45 Rui gave no rise in the first 5 hours and only 4 cm. rise after 5 days. There was no rise of capillary water at all after treatment even after five days.

Capillary rise of water before and after leaching indicate that soidumization of the soil had taken place after leaching.

The capillary rise in the original soil of S. No. 101 was 1·5 cm. after five hours and 10 cm. after five days. This was evidently due to salts. In the sample after treatment, there was no rise in five hours and slight in five days presumably because the salts were not completely reached out and some salt was still present. A third sample taken from the inside of the canal bank was examined for comparison.

In this case the capillary rise was moderate (6·5 cm.) in five hours and fairly high in 24 hours.

Soils from S. No. 45 and 101 of Rui, sodiumised by the leaching process and the sample taken from the side of the canal in flowing water near Rui were subjected to percolation test. These samples were packed in glass tubes one inch in diameter and 12 inches long to a length of 8 inches with a previous layer 1½ inch of *murrum* on either end. An average head of 5 feet that usually occurred in the canal was maintained. There was no percolation at all through the sodiumised soil for 5 feet head nor for the 9 feet head. Percolation through the soil from the canal bank was 9·2 cc. per day for 5 feet head and 23 cc. per day for 9 feet head.

These results on calculation showed that percolation from the canal bank at present with 5 feet head was 0·69 cusecs per million square feet. Observations taken in 1918 put this at 1·06 cusecs per million square feet.

Sodiumised soil can be used for lining the canal to prevent percolation. There is no need to sodiumise the soil by treatment with any salt; natural sodium soil exists in the locality over a large area in the close vicinity of the canal. This sodium soil can be used as it is and will not cost any thing extra besides labour charges The canal bank section should be removed to a depth of 13 inches. The exposed surface should be lined with sodium clay by forcibly throwing lumps of puddled clay thereon. The clay should then be covered with a casing of *murrum*, four inches in thickness.

(3) EXPERIMENTS CONTINUED WITH STAUNCHING OF CANALS WITH *SURKHI* MORTAR (3)

Experiments with *surki* mortar concrete lining were continued. Two of jelley to one of *surki* mortar four inches thick and one square feet area was subjected to a head of 16 feet clear water. The seepage losses through concrete lining is practically steady year after year as given in Table 13 C. 14.

TABLE 13 C. 14

Date	Percolation per million square feet in cusecs
September 20, 1947	1·60
September 27, 1947	1·57
October 11, 1947	1·60
December 9, 1947	1·15
December 25, 1947	0·99
January 5, 1948	1·00
January 26, 1948	1·14
February 10, 1948	1·00
February 24, 1948	1·17
March 4, 1948	1·36
March 22, 1948	1·28

DISCUSSION BY THE RESEARCH COMMITTEE

MR. S. L. MALHOTRA introduced item (1) and said that they were still carrying on experiments. Nothing conclusive had yet been found.

DR. V. K. GOKHALE introduced item (2). A large area of about 600 acres lay along the Godavari Left Bank canal between miles 8/7 to 10/5 which was damaged by waterlogging and salt and damage was increasing gradually. There was no irrigation and the damage was due mainly to leakage from the

(3) Hydraulic Research Station, Krishnarajasagar, Annual Report 1947, page 68.

canal which cut through *murrum*. This area could not be drained as the soil was deep and there was no pervious layer within ten feet. An estimate was framed and sanctioned by the Bombay Government in the 1942 for staunching the canal. It was proposed to use sodium carbonate for preparing a sodium clay puddle. On account of the war sodium carbonate was not available in quantity required and within the amount estimated. An attempt was, therefore, made to see if sodium claly occurred under natural conditions in the vicinity of the canal. This had been found in the damaged area itself. This soil was highly sodiumised and was water-tight.

It was now proposed to line the canal, for short lengths with this sodium clay and some other lining materials such as soil cement, *etc.*

MR. N. S. GOVINDA RAO introduced item (3).

DR. R. C. HOON referring to the use of sodium clay said that experience in the Punjab had not been very happy. With passage of time, there must be a certain amount of deterioration. He wondered whether experiments had been tried for a longer time.

RAI BAHADUR KANWAR SAIN said that on the Bikaner canal during the last two years, the seepage losses had doubled. He wondered if the Punjab would take up investigations into this. There were some minor cracks, but nothing which would be termed very serious. Before the last floods a lot of damage was done, but it was repaired.

THE PRESIDENT suggested that it would he better if he carried out some independent checks.

THE SECRETARY enquired if he was sure that the discharge table in use, which showed the losses to have been doubled, was correct.

RAI BAHADUR KANWAR SAIN replied that at the lower end there was a meter flume and at the head the discharge was observed by the headwork division.

THE SECRETARY pointed out that the discharge site was not in the *pacca* reach.

MR. S. L. MALHOTRA enquired about the magnitude of the losses.

RAI BAHADUR KANWAR SAIN replied that in summer the losses had risen from 150 to 300 cusecs and in winter too they had been doubled.

DR. J. K. MALHOTRA enquired as to what the figure per million square feet of wetted surface worked out to.

THE SECRETARY enquired about the distance upto which the damage extended in the canal.

RAI BAHADUR KANWAR SAIN replied that it extended upto R. D. 47,000 and there was one breach at R. D. 75,000.

DR. R. C. HOON said that he carried the analysis of the canal water from the head of the tail reaches of the Bikaner Canal some years back, the results showed definite indication of a higher column contents in the water samples from the tail reaches. Probably, the continuation of the process of dissolution of time from the material composing the canal lining might account for the increase in the canal losses.

RAI BAHADUR KANWAR SAIN said that most of the canal lay in the Punjab. In fact he had suggested to Rai Bahadur Gita Ram Garg that it would be worthwhile carrying out a detailed investigation not only for sake of the Bikaner Canal, but also for guidance in designing the lining of the future canal. The matter was one which the Punjab Irrigation should take up. Bikaner would give all the help that was needed. Bikaner had not made an official reference yet.

MR. L. K. MITTAL said that they also had carried out some experiments in the United Provinces on canal linings. They got the very same results as did the Punjab. The reduction in losses in the first year was 85 per cent., but at the end of one year, it was 55 per cent., and experiments were carried out for another three years and losses came down to 50 per cent. Then the Chief Engineer ordered to stop experiments because the lining was deteriorating. In such a case sodium carbonate would hardly help. In case of Poona soil there was a high percentage of clay which would ordinarily be water tight. One does not need to bother about sodiumisation of the soil as the sodiumisation is bound to deteriorate and the soil is water tight even without this.

DR. V. K. GOKHALE said that they proposed to use sodium clay. They were going to do detailed experiments on that. They had carried out only laboratory tests till then.

THE PRESIDENT suggested that it would be quite good to take up time tests in the laboratory.

DR. R. C. HOON suggested that the tests should be carried out on a minor.

RAI BAHADUR C. L. HANDA referring to losses in the Bikaner Canal enquired whether there had been any change in the establishment responsible for water measurements.

RAI BAHADUR KANWAR SAIN replied that there might have been such a change. He could not say definitely.

It was then decided to keep the subject on the agenda.

DISCUSSION BY THE BOARD

THE SECRETARY said that three items were discussed at the Research Committee Meeting (page 982). There was no resolution.

(iii) ECONOMICS

PRELIMINARY NOTE

There was no contribution or discussion under this sub-head at the 1947 Research Committee Meeting.

Recent Literature.

(1) Young W.R.—Low cost linings for irrigation canals—Indian Concrete Journal, Vol. 21, No. 10, October 1947.

THE YEAR'S WORK

The following item was discussed at the 1948 Research Committee Meeting :—

(1) Most economical section for a lined channel.

(1) MOST ECONOMICAL SECTION FOR A LINED CHANNEL [4]

The staunching of irrigation channels has three aspects, each of which requires careful consideration *viz.*, (*i*) the shape of the lined section, (*ii*) strength and permeability of the lining material used, and (*iii*) the cost of lining in relation to the volume of water saved.

The second and third aspects respectively involve tests on the lining under different water heads and estimation of the cost of lining per unit surface area, and have been the subject of several investigations. The first has, however, received closer attention only in recent years, and would seem to repay further examination.

A lined channel has usually a smaller section than one which is not lined and carries an equal discharge. This is due to lower rugosity of the lined section, which enables it to carry, for the same depth and slope, its supply with a higher velocity and lesser water width, than the unlined section.

While the resulting saving in excavation and land acquisition is one of the obvious advantages of lining, the question of using the reduced waterway to still better purpose is one which has not yet been fully investigated.

The section adopted for the Bikaner canal, the first large channel to be lined in Northern India, was of the usual trapezoidal form, with a flat bed and sloping sides. The side slope was not uniform, the final section being somewhat of the form shown in Figure 13 C. 5.

The design of the Haveli lined canal was also based on a trapezoidal form, but instead of the side slopes meeting the bed at an angle, a curvilinear transition was provided Figure 13 C. 6.

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(4) East Punjab Irrigation Research Institution, Amritsar, Annual Report 1947 Pages 57—61.

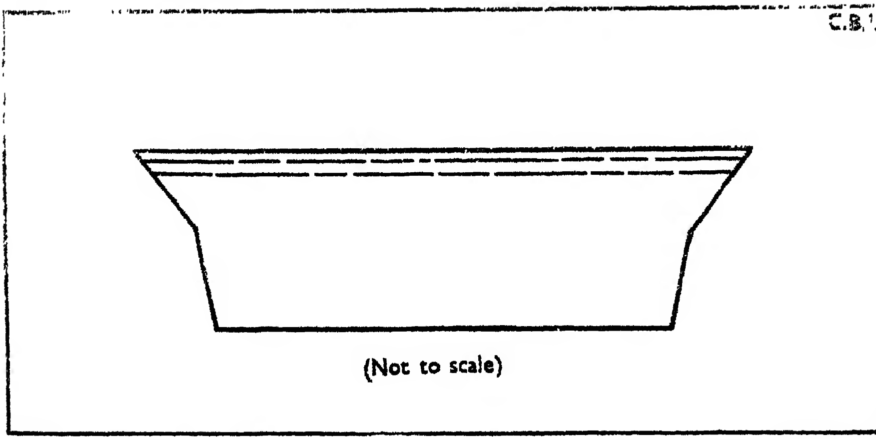

Figure 13C.5:-*Showing section of lined Bikaner Canal*

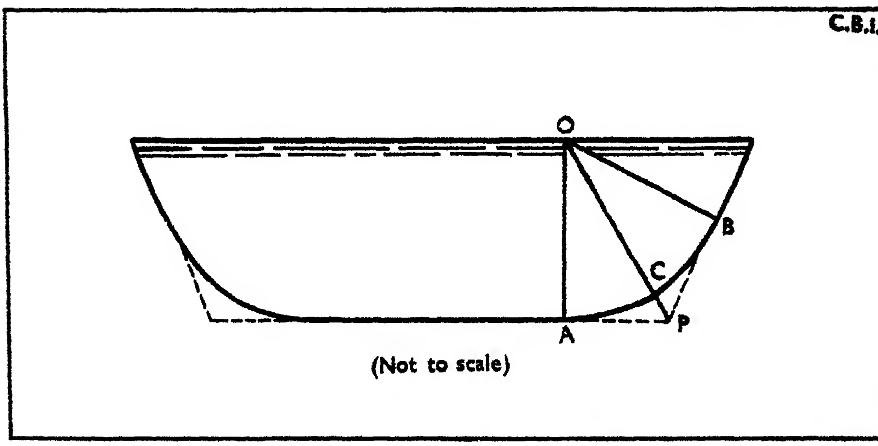

Figure 13C.6:-*Showing section of lined Haveli Canal*

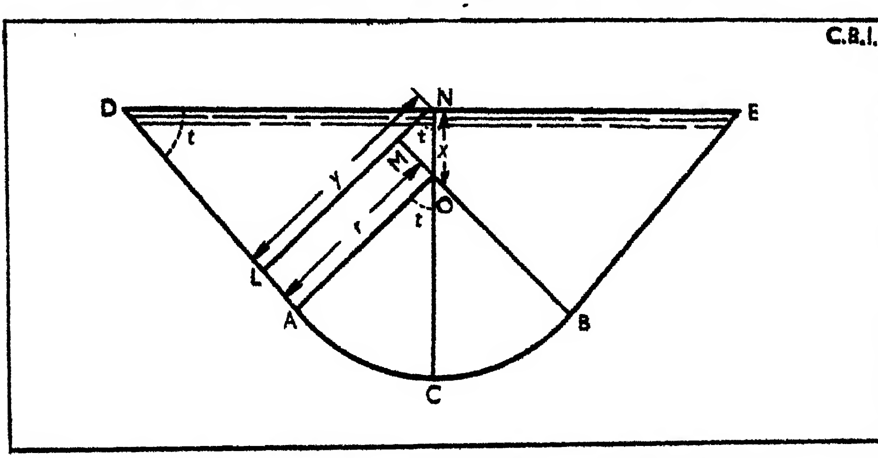

Figure 13C.7:-*Showing Haigh type lined section*

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The curve ACB, which replaced the portion APB, was the arc of a circle, and its use was believed to obviate the necessity of a cut-off at P, (which would otherwise have been necessary, in order to reduce the uplift on the lining, the water table in some reaches being higher than the bed) and to provide a better support for the weight of the lining on the slope above B than that likely to be given by an angular joint.

The question of the most economical section was apparently first studied by Haigh. He advocated, for channels carrying less than 2,000 cusecs, a section formed by the arc of a circle, joined tangentially to straight side slopes the latter being adjusted to the angle of repose of the soil.

The theory of the 'Haigh' type section was elaborated in the Institute during 1947. The Haveli type was under examination at the close of the year.

The Haigh type section consists of Figure 13 C. 7 :—

(*i*) The arc ACB of a cricle of radius r, with its centre at O. The angle AOB equals $2\ t$.

(*ii*) The side slopes AD and BE, which are tangents to the circle at A and B. The angles they make with the horizontal, each equals t.

If O does not lie in the water surface, let it be at a depth x below it, N being the point vertically above O. Let y be the length of the perpendicular from N on AD.

Then

the wetted perimeter, P = arc ACB+2 AD

=arc ACB+2 (AL+LD)

$=2\,rt+2\,(x \sin t+y \cot t)$

$$\text{Now } y = \text{NL}=\text{NM}+\text{ML} \qquad (13\text{C}.\ 1)$$

$$= x \cos t+ r$$

$$\therefore x = (y-r)/\cos t \qquad (13\text{C}.\ 2)$$

Substituting from (13C.2) in (13C.1)

$P=2\ rt+2\ (y.\ r.\ \tan t+y \cot t)$

$$=2r\ (t=\tan t)+2y\ (\tan t+ \cot t) \qquad (13\text{C}.\ 3)$$

The area of the waterway,

A=sector OAB+2 (trapezoid ONLA+triangle NLD)

$=r^2t+(y+r)\,x\sin\ t+y^2\cot t$

Substituting from (13C.2)

$=r^2\,t+(y^2-r^2)\tan t+y^2\cot t$

$=r^2(t-\tan\ t)+y^2(\tan t+\cot t)$ .. .. .. (13C. 4)

Putting $a=t-\tan t$

$b=\tan t+\cot t$

We get $P=2\,(ar+by)$ .. .. (13C. 5)

$A=a^2r+by^2$

$\therefore\ A=\frac{1}{a}\left(\frac{P}{2}-by\right)^2+\ by^2$ .. .. .. (13C. 6)

for a given perimeter, P, and known side slopes, therefore,

A is a maximum when

$0=dA/dy=\frac{2b}{a}\,(P/2-by)+2\,by$

or $P=2\,(a+b)\,y$ .. .. (13C. 7)

Also $P=2\,(ar+by)$ .. .. (13C. 5)

$\therefore\quad y=r$ .. .. (13C. 8)

Hence $a=ar^2+by^2$

$=(a+b)\,r^2$ .. .. (13C. 9)

and $P=2\,(a+b)\,r$ .. ..(13C. 10)

Equation (13 C. 8) shows that *the area of the Haigh type waterway is a maximum ratio when the centre of the circular portion lies in the water surface.*

Also substituting the values of a and b in (13C.9) and (13C.10)

$A=(t+\cot t)\,r^2$

$P=2\,(t+\cot t)\,r$

Hence $R=A/P=r/2$ .. .. (13C. 11)

This shows that, *for the Haigh section.*

(*i*) *the mean hydraulic radius, R, equals half the maximum depth and is independent of the side slopes, and*

(*ii*) *the ratio, R/P, which defines the 'share' of the section* is independent of the dimensions of the section.

The values of maximum A/r^2 for different side slopes come out as follows :—

t (degrees)	A/r^2
30	2·26
45	1·79
60	1·62
90	1·57

The most economical section may, however, be defined as the section which has the maximum area of waterway, for a given surface width and a given wetted perimeter.

Theoretically as will be shown in later reports, such a section is the arc of a circle passing through the end points of the water surface.

The Haigh type is, therefore, not the most economical section possible, though, of the type containing a circular arc as the base with a straight side slope, the Haigh section is the best.

Again if we consider Lacey's v-f-R relation

$$V = 4 f R/3,$$

we get, for the Haigh section,

$$V^2 = 2 fr/3 \text{ (since } R = r/2).$$

Also $Q = AV$

$$= (t + \cot t)\, r^2 .\, (2fr/3)^{\frac{1}{2}}$$

$$r = (1 \cdot 5 f)^{1/5} (Q\ t + \cot t)_{\overline{5}}$$

With $f = 1 \cdot 0$ and $t = 45°$, as advocated for ordinary use by Haigh, we get,

$$r = 0 \cdot 86\, Q^{2/5}$$

This gives, for values of Q equal to 1,000 ; 2,000 ; and 4,000 cusecs, values of r equalling 13·6, 18·0 and 24·8 feet respectively. As some of the ordinary linings, when tested under heads exceeding about 15 feet, either give way or allow appreciable seepage losses, it would seem that the Haigh type section should preferably be used only for discharges up to about 1,200 cuses, and not be adopted unreservedly on larger canals. This might be taken as another limitation on its use.

Another point in regard to the utility of any lined section is the practicability of constructing curvilinear portions. While a curved section may, theoretically, be the most economical, ease of construction has also to be considered, as the margin of economy gained one way may be offset by the extra time and labour involved in putting in curved transitions.

This aspect is one for practical engineers to consider. The necessary data for comparison will, however, be provided by theoretical studies.

DISCUSSION BY THE RESEARCH COMMITTEE

Dr. J. K. Malhotra introduced the item. He said that there had been a nag in what he had prepared, and he would correct the same in the next annual eport.

There was no discussion and it was decided to keep the subject on the genda.

DISCUSSION BY THE BOARD

The Secretary said that there was no discussion at the Research ommittee Meeting.

As regards proposed C.B.I. Publication on 'Staunching of Canals,' Dr. K. Malhotra of the Irrigation Research Institute had undertaken the preration of the draft. A bibliography of all the literature on the subject had nce been supplied to Dr. Malhotra.

Dr. J. K. Malhotra gave a brief idea of the work being done by him in der to evolve the 'most economical' section for a lined channel.

He stated that usually the unlined channels were given a trapezoidal secon and the same shape was adopted for the lined channels, as in the case of e Bikaner Canal and the power channel of the proposed Yamuna Power heme. A departure from this shape was made in the case of the Haveli ed canal, where the bottom corners of the trapezoid were rounded off. This s useful in that the resulting arch action provided a support for the lining laid er the side slopes and obviated the necessity for a toe-wall at the joint of the d and side slopes.

Continuing, he remarked that the main question before him was to evolve a Haveli-Type section; which, for a given discharge and slope, would have the least perimeter, as the cost of lining was obviously a function of the total perimetral area. This really boiled down to a choice of a suitable bed width : depth ratio.

A section could be even ' bedless ', in which case it would be very deep and narrow; or it could have a relatively large bed-width when it would be flat and shallow. Without going into the details of mathematics, it appeared that something mid-way between these extremes might give the best results.

He added that he could give no exact idea of the order of economy possible with such a section, but it seemed that substantial savings could be effected provided sections somewhat deeper than usual were adopted.

The problem was still very far from final solution, as so many other factors besides shape were involved. For instance, there was the question of the rugosity co-efficient. For the Haveli Canal, a rugosity of 0·0150 was assumed for design; but the actual observations, after the canal had run for a year or so, showed that it had gone up to 0·0165. On the Nangal Project, the power channel was being designed to a co-efficient of 0·0180; and the adoption of as low a value as 0·0125 in the Yamuna Power Scheme would seem to need reconsideration. He thought that it would be safest to fix the rugosity co-efficients after carrying out model experiments with the actual materials to be used in canal lining.

Another item on which research would be essential was to find what safe velocity the lining material would stand.

The East Punjab Research Institute hoped to look into all these things and might have something to say at a later date.

14C. Grouting

PRELIMINARY NOTE

This subject was introduced on the agenda for the first time in 1947 with the following sub-heads :—

(*i*) Materials, (*ii*) of foundation, (*iii*) for staunching.

(i) Materials

There was no contribution or discussion under this sub-head, at the 1947 Research Committee Meeting.

(ii) Grouting of Foundations

There was no contribution or discussion under this head, at the 1947 Research Committee Meeting.

Recent Literature.

(1) Ischy E., Ingenieur E. P. Z. Directeur general de l'Entreprise de Foudation et Travaux Hydrauliques France.—Barrage de Castillon—Lutte Contre les erosions souterraines (Castillon Dam—fighting underground erosions)—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 36.

(2) Ischy E., Ingenieur E. P. Z. Directeur general de l'Entreprise de Fondations et Travaux Hydrauliques France.—Digue du Lac Noir—Lutte Contre les erosions souterraines (Lac Noir Dam—fighting underground erosions)—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 37.

(iii) Grouting for Staunching

There was no contribution or discussion under the above sub-head at the 1947 Research Committee Meeting :—

Recent Literature.

(1) Gysel, Gottfried, Ing. dipl. E. P. F., S. A. de l'Usine de Rupperswil—Auenstein and Blatter Charles, Ign. dipl. E. P. F. Swissboring, Societe Suisse de Sondages et Prospections S. A.—Etanchement de renards par injection d'un gel d'argile a la digue de l'AAR de l'usine Hydro-electrique de

Rupperswil—Auenstein (Caulking of pipings by grouting of frozen clay at AAR embankment Rupperswil—Auenstein hydro-electric power station)—International Commission on Large Dams, Third Congress, Stockholm, 1948, R 31.

THE YEAR'S WORK

DISCUSSION BY THE RESEARCH COMMITTEE

There was no contribution on the subject.

It was decided to keep the subject on the agenda.

DISCUSSION BY THE BOARD

The Secretary said that there was no contribution and no discussion on the Research Committee Meeting under these sub-heads.

15C. Tubewells

PRELIMINARY NOTE

In 1947 the subject of Tubewells was divided into two parts, Sub terranean waters and Tubewells.

The sub-heads falling under the former have been taken under Section C—Hydrology—item 7. The latter is divided into six sub-heads.

(i) Types

The following item was discussed at the 1947 Research Committee Meeting:—

(I) Radial tubewells.

Recent Literature.

(1) Khanna R. K.—Lift irrigation—Krishnapura type persian wheel—Central Board of Irrigation Journal, Vol. 3, No. 3, July 1946.

THE YEAR'S WORK

The following items were discussed at the 1948 Research Committee Meeting :—

(1) Radial tubewells.

(2) Feeder tubewells.

(1) RADIAL TUBEWELLS [1]

Basic study of the radial well was taken up last year and the effect of length and number of radials as also the location of the radials in the strata were studied. This year the effect of staggering the radials in two planes within the strata of varying depths has been investigated.

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[1] United Provinces Irrigation Research Station, Annual Report, 1947, pages 85-86.

The model was built to scale one foot=0·5 mm., the radials being 200 feet long and the central sump 16 feet in diameter. The apparatus is shown in Figure 15 C.1.

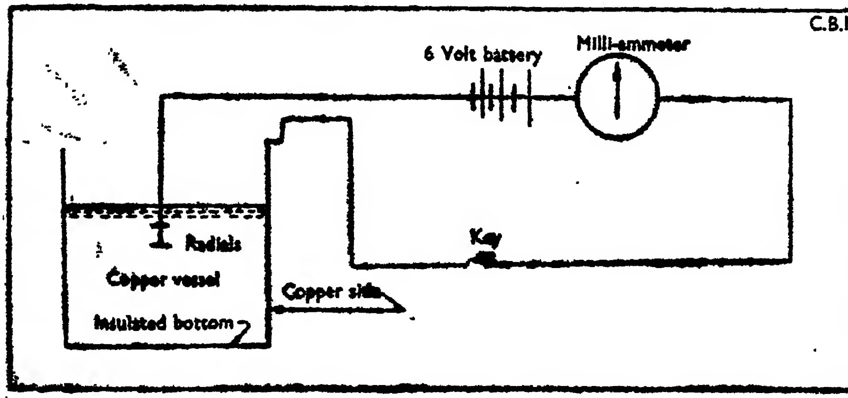

Figure 15 C.1 : *The Electrical analogy method showing staggering of Radials.*

Four radials two in each plane, were placed and their discharges with 50 feet, 75 feet, 100 feet and 200 feet depth of strata were observed. In each case the distance between the planes was varied in order to get maximum discharge. Table 15 C.1 below shows the results.

TABLE 15 C.1

Serial No.	Depth of the strata in feet	Discharge without staggering in cusecs	Increase in discharge due to staggering in cusecs	Percentage increase
1	50	53·5	1·5	2·8
2	75	60·0	0·5	0·9
3	100	64·5	2·5	3·9
4	200	74·5	1·5	2·0

It is evident that staggering while it introduces many constructional difficulties does not add to discharge materially.

A design in which the addition of a vertical tube to four radials in one horizontal plane was proposed and tested. The proposal was tested on radials each 200 feet in one horizontal plane and a vertical tube of 50 feet length. The strata was taken as 100 feet deep.

Following are the observations on the above design as a result of experiments on electrical analogy apparatus. The advantage of adding a vertical radial looked meagre in view of the fact that it would be surrounded on all sides by the cone of depression created by the radials.

(*a*) Discharge in milliamperes with four radials, the location of the radials being in the centre of the strata =29.

(*b*) Discharge of above with an addition of 50 feet vertical tube below the radials=30.

The increase in the discharge of the order of 3·4 per cent. is insignificant as compared to the cost of the vertical strainer. Hence it is not recommended.

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(2) FEEDER TUBEWELLS ([2])

ABSTRACT

Since the construction of the 30 feeder wells of three cusecs each along the Ganges canal near Meerut, located at distances varying from 450 feet to 1,200 feet from the canal bank, a controversy has been rife as to their real utility in augmenting the supplies of the canal. In 1937 field experiments were launched on these wells to determine the increase in seepage from the canal by boring the pipes on lines radiating from the wells both towards and away from the canal. Observations of the water table were made after running the wells in the first instance for a prolonged period and then the reaction of the water table a week after the wells were stopped was watched.

The observations have been studied previously by many persons, but no quantitative idea of any sort was formed except that for the qualitative idea that recoupment of the supplies from the canal side was greater as compared to that from other directions. This consclusion was reached on account of the steeper gradients of water table on the canal side.

Further efforts were made to see if any useful result could be arrived at from the already collected experimental data before any fresh field experiments are taken.

A fresh approach has, therefore, been made to analyse this incomplete and insufficient data.

Figures 15 C.2, 15 C.3 and 15 C.4 show the equipotential lines of the water table around the wells No. 3, 16 and 29 as obtained from the experimental data. These wells were situated at distances of 544, 660 and 1,143 feet respectively from the edge of the canal.

([2]) United Provinces Irrigation Research Station, Annual Report 1947, pages 86-90.

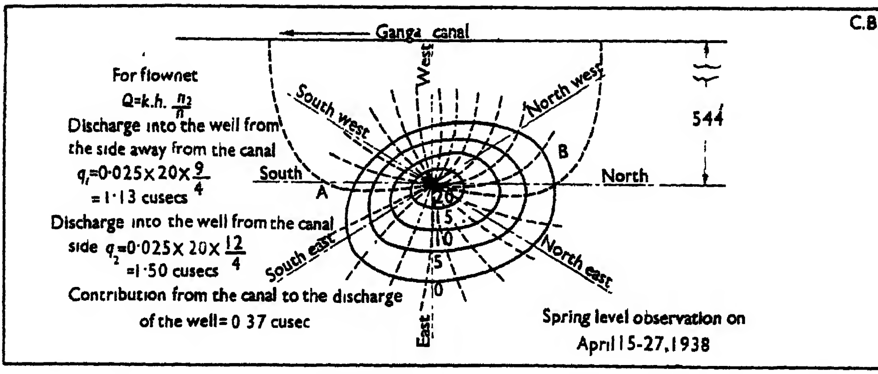

Figure 15C.2:-*Showing flownet analysis on Feeder Well No. 3*

Scale 400 200 0 400 800 1,200 Feet

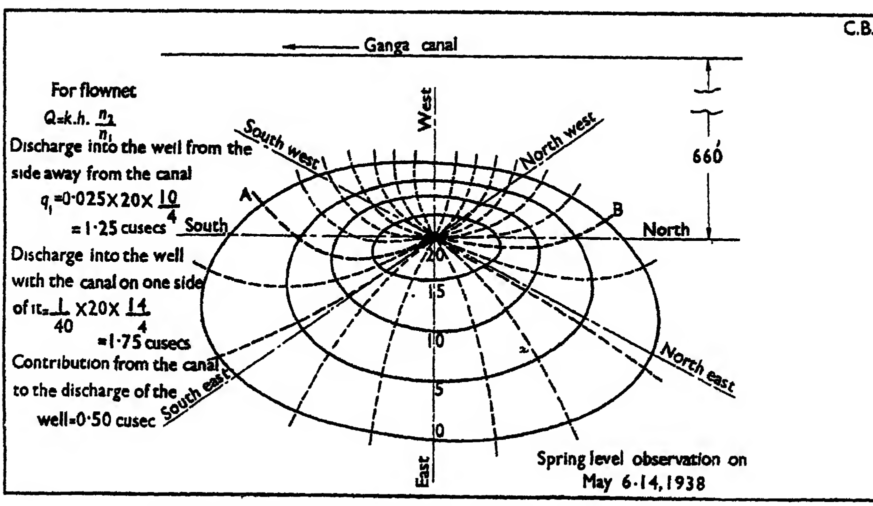

Figure 15C.3:-*Showing flownet analysis on Feeder Well No. 16*

Scale 400 200 0 400 800 1,200 Feet

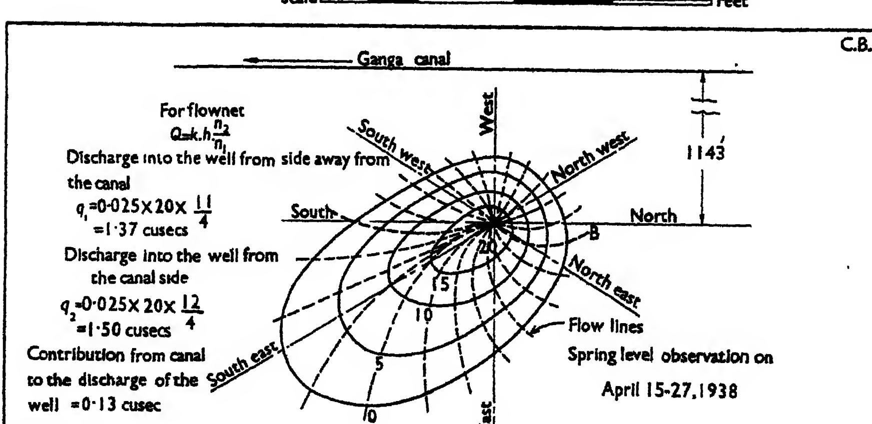

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Figure 15C.4:-*Showing flownet analysis on Feeder Well No. 29*

Scale 400 200 0 400 800 1,200 Feet

C.B.I.

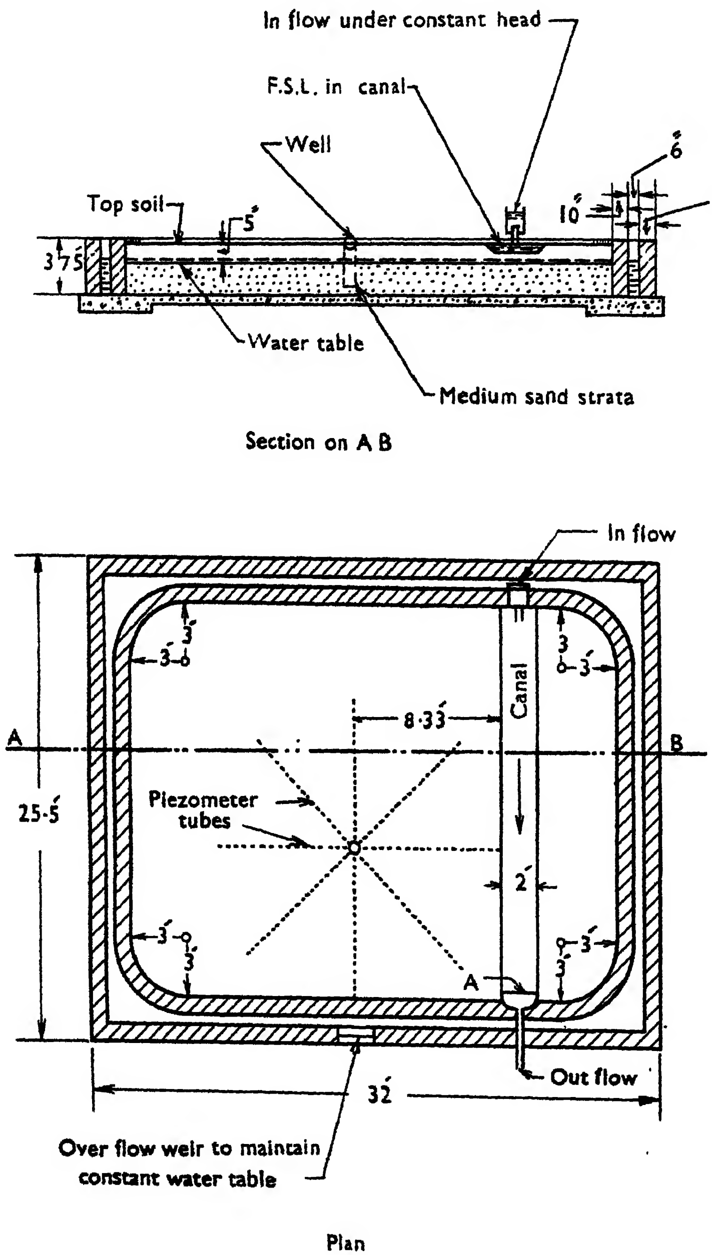

Figure 15C.5:- *Showing feeder well model*

Scale 10 5 0 10 20 Feet

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It is evident that some quantitative idea must be formed to decide this issue. If the feeder well at a particular distance were to draw only a small part of its supply from a short length of the canal it should not matter much. On the other hand if the seepage losses in a considerable length of the canal would be materially increased the well should be removed to a safer distance.

In order to have a quantitative idea from the data supplied the average value of the co-efficient of permeability has been assumed to be 0·025 and a flow net constructed around the tubewells as shown in Figures 15 C.2, 15 C.3 and 15 C.4. From this flow net we can assess the amount of water that is flowing into the well from different directions. The strata of the well has been divided into two portions (*a*) the area lying on the canal side where the flow lines indicate the water flowing from the canal in addition to the sub-soil and (*b*) the area lying in the other side *i.e.*, on the side away from the canal. It should be noticed that on one side we are getting water from the canal and the sub-soil while on the other side we are getting water only from the sub-soil. The difference of the two, therefore, should give us the contribution of the canal to the well.

For a flow net we have the discharge

$q = k \times h \times n_2/n_1$, where

q is the discharge in cusecs

k is the co-efficient of permeability

h is the depression

n_2 is the number of squares between two adjacent equipotential lines

n_1 is the number of squares between two adjacent flow lines

Table No. 24 gives the results with

$k = 0{\cdot}025$ and $h = 20$ feet.

It will be seen from Table 15 C.1 that in the case of wells No. 3, 16 and 29 the contribution of the canal to the tube well discharge comes out to be 0·37, 0·50 and 0·13 cusec.

These results, as will be noted, are based on certain approximations and assumptions, so their accuracy in the quantitative sense is rather limited ; however the trend and the order of magnitude of the contribution by the canal to the tubewell discharge cannot be much different.

In order to study further the problem it was decided to carry on the experiments in the laboratory. A model to scale 10 inches=one foot was constructed. This is shown in Figure 15 C.5 and is self explanatory. The average strata of the top layer containing the canal which is a loam giving seepage losses of the

TABLE 15 C.1

Serial No.	Tube-well No.	Canal side				Other side			Difference col. 6—col. 9, i. e. contribution from the canal to the discharge of the well in cusecs	Increased seepage of canal of the affected reach due to tube-well	Extra losses evaluated as percentage of tubewell discharge	Remarks (Trend taken from Table 25 Column 7.)
		Distance of the canal from the well in feet	N_2	N_1	Discharge $q=k.h.\frac{N_2}{N_1}$ cusecs	N_2	$N1$	Discharge $q=k.h.\frac{N_2}{N_1}$ cusecs				
1	2	3	4	5	6	7	8	9	10	11	12	13
1	3	544	12	4	1·50	9	4	1·13	0·37	0·21	8·00	Result *vide* S. N. 5.
2	16	660	14	4	1·75	10	4	1·25	0·50	0·10	3·30	Result *vide* S. N. 6.
3	29	1,143	11	4	1·38	10	4	1·25	0·13	0·04	1·50	Result *vide* S. N. 10.

Note.—Columns 11 and 12 obtained from the trend of laboratory experiments *vide* Table No. 25 and column 16 as per Remarks column 13 above.

order of 4 to 5 cusecs per million square feet of wetted perimeter—as obtained in canals of similar size—was reproduced in the model. The rest was made in good coarse and similar to that obtained in tubewells at spots where strainers are located.

Water at a constant rate was fed into the canal. Part of the water seeped and part escaped over the weir A, Figure 15 C.5, meant to maintain a constant depth of water in the canal. The escaping water was collected and measured. The difference of the incoming water and escaping water gave the seepage losses in the canal. A tubewell made up of strainer wire mesh was sunk at different distances from the canal. Water was pumped out of the well at a constant rate so as to maintain a depression of 20 feet. The difference in the in coming water and outgoing water again gave the seepage losses. The difference of the two losses gave the increase in seepage from canal due to tubewell. The whole model was kept covered with wet gunny bags to maintain constant temperature and humidity to eliminate weather effects.

Perforated pipes were also sunk along eight lines radiating from the well to plot the water table contours and to know the length of the canal affected by the draft from the tubewell. Water table contours for different positions of the tubewell with respect to the canal were plotted. In order to compare the results of the laboratory experiments with the field results, calculations were made by drawing the flow nets for the contribution of the water from the canal to the tubewell. These are tabulated in Table 15 C.2 . It will be noticed from Table 15 C.1 and 15 C.2 that there is a fair resemblance in the results of the field and laboratory experiments.

The effect of the tubewell on the canal losses is tabulated in Table 15 C.2.

CONCLUSIONS

From the laboratory experiments and those done in the field, as analysed above, it is clear that the increase in losses of the canal is dependent on a number of factors the most important ones being :—

(*i*) Difference between the water table and the canal full supply level,

(*ii*) The distance of the tubewell from the canal,

(*iii*) The depression in the tubewell,

(*iv*) Strata connection between the strainer and canal bed.

Item (*iv*) is seldom possible in this province due to a good covering depth of loam always met with. With the data in hand, an approximate quantitative idea of the losses can be formed for a water table which may be roughly 25 feet below full supply level of the canal, with an average depression of 20 feet. This is usually obtained at three cusecs feeder well and was also maintained in the

TABLE 15 C.2

Result of the feeder well experiments.

Serial No.	Depth of water table below the bed of the canal in feet	Distance of the tubewell from the edge of the canal in feet	Losses from the canal in cusecs per million square feet wetted perimeter	Length of the canal effected (from drawings 28-47 in feet	Losses from the reach of the canal effected (without tubewell) (cusecs)	Increase in losses as observed	% increase as observed $\frac{\text{Col. 7}}{\text{Col. 6}}\times 100$	From the flow net *vide* Drawings 28-47: Discharge from the canal side N_s	N_1	Q_1	Discharge from the other side N	N_1	Q_2	Difference col. 11-col. 14 contribution from canal	Increase in seepage of canal due to tubewell as percentage of contributions from the canal $\frac{\text{Col. 7}}{\text{Col. 15}}\times 100$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	15
1	50	400	4·77	2,000	2·19	0·36	16·43	41	16	1·70	13	16	0·55	1·15	31·4
2	25	400	3·10	1,300	0·953	0·18	18·80	27	16	1·60	11	16	0·65	0·95	18·9
3	15	400	3·08	1,280	0·902	0·075	8·3	24	16	1·25	16	16	1·00	0·25	30
4	5	400	2·67	1,450	..	..	..	13	16	1·17	12	16	1·08	0·09	0
5	50	600	4·07	1,800	1·68	189	9·4	17	16	1·32	12	16	0·93	0·39	48·5
6	25	700	4·54	1,950	1·95	0·088	4·5	20	16	1·36	13	16	0·89	0·47	18·8
7	15	700	3·96	1,700	..	..	..	15	16	1·16	14	16	1·09	0·07	..
8	5	700	4·24	1,710	..	..	..	15	16	1·20	13	16	1·05	0·15	..
9	50	800	4·67	2,300	2·47	0·0835	3·28	18	16	1·25	15	16	1·00	0·25	33·5
10	50	1,000	4·97	2,400	2·74	0·0460	1·68	15	16	1·25	13	16	1·05	0·15	3·06
11	25	1,000	4·96	..	..	..	..	..	..	..	..	..	..	..	..
12	15	1,000	4·27	..	..	..	..	..	..	..	..	..	..	..	..

laboratory experiments. For these conditions it seems that at a distance of 660 feet the extra losses created in the canal by the tubewell are 0·1 cusec only against 3·0 cusecs extracted, although the contribution from the canal (due to its natural losses) is of the order of 0·4 cusec which is only intercepted by the tubewell. Figures for a distance of 1,143 feet give 1·5 per cent. of the tubewell discharge on account of the increase in seepage created by the presence of the tubewell.

From this it seems clear that the optimum distance between the canal and the tubewell for the condition of water table depression *etc.*, referred to above, can be considerad as about 1,000 feet, when the increase in seepage can be considered as insignificant compared to the advantage of the additional discharge obtained from the sub-soil. The optimum spacing of such wells for the conditions as at 30 feeder wells of the Upper Ganges canal can be taken as about four furlongs. The above quantitative results are based on certain approximations and assumptions—laboratory results have been made use of for calculating the per centage increase of losses *etc.*, in the field—but the results are dependable in so far that they cannot be widely out. If, however, a more exact idea is to be obtained, field experiments can be launched with scme hope of success with this much experience in hand. Piezometer pipes would require to be placed and observed before a new feeder well is put into operation, and thereafter, the exact reaction of water table can be studied under controlled conditions. It would not be advisable to perform these experiments on the existing tubewells where the water table is already vitiated by years of working.

DISCUSSION BY THE RESEARCH COMMITTEE

Mr. L. K. Mittal introduced both the items and said that basic studies had been continued during the year.

Rai Bahadur M. C. Bijawat referring to the types of tubewells said that so far they had been constructing two types—Strainer type and Casing type. Recently they had introduced a third type. In this the strainers had been replaced by ordinary steel tubes with slits made three inches apart and shrouded by gravel. This was merely based on rule of thumb and not on any experiments or theory. They had found that from the same depth of tubewell they got a discharge of 40 thousand to 47 thousand gallons with a depression head of only 6 to 10 feet in Bihar. The reason for increase in discharge appeared to be that in this type the whole strata was tapped as slotted tubes were placed through out the depth of the well. Observations had shown that even in some of the clays there was a pressure of water and all this was drawn through the slots in the tubes.

Another reason was that shrouding increased the diameter of the tubewell and the result was an increase in discharge with less depression.

They, however, required far more data to be more definite about the thing. Further investigations would have to be made. Sizes of slits and the distance between them would have to be adjusted according to the type of strata based upon proper investigations. The results obtained in Bihar, however, held out strong hopes that it might be possible to draw more water at lesser depression heads.

DR. V. I. VAIDHIANATHAN said that non of them would come to any conclusion. Some of the old experiments that did not give any result were being repeated. Experiments were performed on Upper Gugera Branch to find out the extent of the cone of depression with a view to fix the distance at which the tubewells should be located from each other and from the canal so that neither the seepage from the canal nor the sphere of activity of one tubewell was affected by the other. Tubewells were meant to lower the sub-soil water table. The distance for that particular case comes to 650 feet but no general conclusion could be drawn from that. The distance would depend upon the strata, depression head, permeability of soil *etc.*, *etc.* It could be anything from 50 feet to 1,000 feet depending upon conditions.

RAI BAHADUR H. L. VADEHRA said that there was another factor which affected seepage from the canal. That was the sub-soil water table. If it was very close to the surface, then, if water were extracted from the sub-soil, the seepage loss would be more than if the water table were lower. Experiments on the subject were made by Mr. K. R. Sharma. He found a critical depth of the sub-soil water table beyond which the seepage from the canal occasioned by pumping was negligible.

Referring to gravel packed tubewells of the so called Roscoe Moss type *i.e.* tubewells with wide slotted strainers, he said that they carried out experiments at Chuharkana and found them to be a failure. They reached the conclusion that those tubewells would be useful only in coarse strata.

They also studied the effect of varying the type of strainer on twenty tubewells in Lahore but did not find any appreciable difference in discharge in parallel of expanding slots. In one group they used telescopic strainers which were shrouded, but found no difference in discharge. Telescopic type was, however, cheaper and hence advantageous.

DR. V. I. VAIDHIANATHAN said that in such experiments the saturation connection between the sub-soil water table and the canal water should be kept up. If the saturation connection was broken the seepage would be affected.

DR. N. K. BOSE referring to item (1) enquired whether the flow between the canal and the sub-soil water level was a saturated one. The depth in some cases was as much as 50 feet and it was rather doubtful if at such a depth of the water table, the flow between the canal and the water table could be saturated. In the Punjab in some cases it was found that there was no saturation connection and the water table was found to have a hump below the canal.

Referring to item (2) he said that on page 87 of the report reference had been made to Dr. Mackenzie Taylor. As a matter of fact the data sent from United Provinces were analysed by Dr. Bose and Dr. Malhotra.

RAI BAHADUR M. C. BIJAWAT replying to Rai Bahadur Vadehra regarding the failure of gravel-packed wells said that the method adopted was merely a rough one. There was no doubt that there was a certain connection between the size of slots and the size of the shrouding material. This certainly needed to be a little bit of coarser grain. It would be the job of DR. Vaidhianathan to go into it. They had just started the experiments.

MR. L. K. MITTAL said that he wanted to put a word of caution in this respect. These experiments were conducted in the United Provinces to assess losses from canals due to tubewell pumping because they were undertaking more feeder well projects. These results only applied to U.P. canals which had been losing water at the rate of 4 to 6 cusecs per million square feet of wetted surface although saturation connection was always there.

It was then decided to keep the subject on the agenda.

DISCUSSION BY THE BOARD

THE SECRETARY said that two items were discussed at the Research Committee Meeting (page 1,008). There was no resolution.

(ii) DESIGN

PRELIMINARY NOTE

There was no contribution or discussion under this sub-head at the 1947 Research Committee Meeting.

Recent Literature.

(1) Peck R. B. and Berman Sidney.—Measurements of pressures against a deep shaft in Plastic clay—International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, 1948, Vol. III.

THE YEAR'S WORK

DISCUSSION BY THE RESEARCH COMMITTEE

There was no contribution and it was decided to keep the subject on the agenda.

DISCUSSION BY THE BOARD

THE SECRETARY said that there was no contribution and no discussion at the Research Committee Meeting.

(iii) CONSTRUCTION

PRELIMINARY NOTE

The following item was discussed at the 1947 Research Committee Meeting under this sub-head :—

(1) Note on boring with 18 inches un-reinforced cement concrete pipes by Mr. Rameshver Saran.

Recent Literature.

(1) Sachs G. and Baldwin W. M.—Folding in tube sinking—Trans. A. S. M. E., Vol. 68, No. 6, August 1946.

(2) Sarwal K. D.—Boring water wells with mechanical drills—Central Board of Irrigation Journal, Vol. 4, No. 4, October 1947.

(3) Tubewells—Letter to the Editor—Central Board of Irrigation Journal Vol. 4, No. 4, October 1947.

THE YEAR'S WORK

There was no contribution.

DISCUSSION BY THE RESEARCH COMMITTEE

RAI BAHADUR C. L. HANDA said that there was an alround shortage of materials, specially steel. About four years ago they first introduced the wooden strainer in the Punjab. It was used in an airfield, as brass strainers were not available. The wooden strainers gave them continuous supply for two years. Later on there was a controversy. In that connection he wanted to say that instances were not known to them about the behaviour of timber put underground. Although controversy was raised, yet he thought there was a good amount of field to cover in further research regarding construction with wooden strainers especially in the shrouded tubewells.

RAI BAHADUR M. C. BIJAWAT said that so far as his information went, perhaps these were not a great success. Further they were not tried sufficiently. It would be possible that Mr. Vaidhianathan would try them. Their life would depend upon the type of water also. Generally speaking wooden strainer would have the same life as the steel strainer in view of the fact that none of our waters were free from salts. The comparative effect of the salts on wood and steel would have to be investigated. This would be borne in mind.

RAI BAHADUR H. L. VADHERA said that he had occasion to use wooden strainers rather extensively in tubewells. They had set up a factory of their own at Chuharkana for manufacturing them. Their experience was that they were useless. They were fragile and difficult to handle. The disharge fell away rapidly.

Dr. V. I. Vaidhianathan said that they would take up the experiments when they started their laboratory.

It was then decided to keep the Subject on the Agenda.

There was no contribution on further sub-heads under Tubewells *viz.*, (*iv*) Extraction of Tubes, (*v*) Operating machinery and their tests, and (*vi*) organisation.

It was decided to keep the subjects on the agenda.

DISCUSSION BY THE BOARD

The Secretary said that there was no contribution under this sub-head. The discussion by the Research Committee was on page 1018 and There was no resolution.

(iv) EXTRACTION OF TUBES

PRELIMINARY NOTE

There was no contribution or discussion under this sub-head at the 1947 Research Committee Meeting.

(v) OPERATING MACHINERY AND THEIR TESTS

PRELIMINARY NOTE

There was no contribution or discussion under this sub-head at the 1947 Research Committee Meeting.

Recent Literature.

(1) Sulzer submersible electric pumps—Sulzer Brothers Ltd., Winterthur, Switzerland, No. 5925 e.

(2) Sulzer borehole pumps—Sulzer Brothers Ltd., Winterthur, Switzerland, No. 5862 e.

(vi) ORGANISATION

PRELIMINARY NOTE

There was no contribution or discussion under this sub-head at the 1947 Research Committee Meeting.

THE YEAR'S WORK

DISCUSSION BY THE BOARD

The Secretary said that there was no contribution under these sub-heads and no discussion at the Research Committee Meeting.

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LIST OF PUBLICATIONS ISSUED BY THE CENTRAL BOARD OF IRRIGATION

(1) Digest of Technical Notes 1931 (Reprint)

A summary of views of Chief Engineers on the following subjects :—

(i) Waterlogging and Reclamation.
(ii) Effect of deforestation and afforestation on river floods.
(iii) Design of works on sand foundations.
(iv) Fluming of canal works.
(v) Regulation at canal head works.
(vi) Design of canal falls.

(2) Digest of Technical Notes, 1932.

A summary of views of Chief Engineers on the following subjects :—

(i) Waterlogging.
(ii) Design of works on sand foundations.
(iii) Design of canal falls.
(iv) Staunching of canals.

(3) Catalogue of books (upto December 1933) in the Library of the Central Board of Irrigation, Simla and its 1st and 2nd Annual Supplements (1934-36).

(4) Hydraulic Diagrams : Revised and enlarged (1948).

(5) Glossary of Technical and Vernacular Terms in connection with Irrigation in India, together with Standard Notations (1934). (1st Revised Edition 1941).

(6) Fluming : A series of examples illustrating the use of Hydraulic Diagrams C.B.I. Publication No. 4 by A. M. R. Montagee M. I. C. E., etc. (1934).

(7) The Standing Wave or Hydraulic Jump (1934). (Second Revised and Enlarged Edition, 1950)

(8) Observation and Record of Pressures below Works on Permeable Foundations, by Shri A. N. Khosla, (1935).

(9) Stabilization of Soils by the use of Bitumen Emulsion—by Brigadier C. H. Haswell, (1935).

(10) Irrigation Canal Falls (1935).

(11) Annual Report (Technical) of the work of the Central Board of Irrigation, India, (1934-35).

(12) Design of Weirs on Permeable Foundations—by Shri A. N. Khosla, Dr. Nolini Kanto Bose and Dr. E. McKenzie Taylor, (1936).

(13) Annual Report (Administrative) of the work of the Central Board of Irrigation, India, (1935-36). (*For the use of Board Members only*).

(14) Annual Report (Technical) of the work of the Central Board of Irrigation, India, (1935-36).

(15) Annual Report (Administrative) of the Central Board of Irrigation, India, (1936-37). (*For the use of Board Members only*).

(16) Annual Report (Technical) of the work of the Central Board of Irrigation, India, (1936-37).

(17) Notes on Waterlogging and land Reclamation in the form of a Questionnaire (1938).

(18) Annual Report (Administrative) of the work of the Central Board of Irrigation, India, (1937-38). (*For the use of Board Members only*).

(19) Annual Report (Technical) of the work of the Central Board of Irrigation, India, (1937-38).

(20) Regime Flow in Incoherent Alluvium—by Gerald Lacey (1939).

(21) Annual Report (Administrative) of the work of the Central Board of Irrigation, India, (1938-39). (*For the use of Board Members only*).

(22) Annual Report (Technical) of the work of the Central Board of Irrigation, India, (1938-39).

(23) Annual Report (Administrative) of the work of the Central Board of Irrigation, India, (1939-40). (*For the use of Board Members only*).

(24) Annual Report (Technical) of the work of the Central Board of Irrigation, India, (1939-40).

(25) Annual Report (Administrative) of the work of the Central Board of Irrigation, India, (1940-41). (*For the use of Board Members only*).

(26) Water weeds and their eradication from canals—by C.C. Inglis, and V. K. Gokhale, with a note on Eradication of Bulrush—by V. K. Gokhale and S. N. Punekar (1941).

(27) Annual Report (Technical) of the Central Board of Irrigation, India, (1941).

(28) Annual Report (Administrative) of the work of the Central Board of Irrigation, India, (1941-42). (*For the use of Board Members only*).

(29) Annual Report (Technical) of the Central Board of Irrigation, India, (1942).

(30) Annual Report (Administrative) of the work of the Central Board o Irrigation, India, (1942-43). (*For the use of Board Members only.*)

(31) Annual Report (Technical) of the Central Board of Irrigation, India (1943).

(32) Annual Report (Administrative) of the work of the Central Board c Irrigation, India, (1943-44). (*For the use of Board Members only*).

(33) Annual Report (Technical) of the Central Board of Irrigation, Indi (1944).

(34) Annual Report (Administrative) of the work of the Central Board c Irrigation, India, (1944-45). (*For the use of Board Members only*).

(35) Annual Report (Technical) of the Central Board of Irrigation, Indi (1945).

(36) Role of Glaciers and Snow on the Hydrology of Punjab Rivers—b Shri Kanwar Sain.

(37) Annual Report (Administrative) of the work of the Central Board Irrigation, India, (1945-46). (*For the use of Board Members only*).

(38) Annual Report (Technical) of the work of the Central Board of Irrig tion, India, 1946.

(39) Annual Report (Administrative) of the work of the Central Board Irrigation India, (1946-47). (*For the use of Board Members only*).

(40) Annual Report (Technical) of the Central Board of Irrigation, Indi 1947.

(41) Resolutions and Recommendations of the Central Board of Irrigati (Technical).

(42) Standards for soil Testing.

(43) Land Reclamation in the Punjab by M. L. Mehta.

(44) Annual Report (Administrative) of the work of the Central Bo of Irrigation, India, (1947-48). (*For the use of Board Members only*).

(45) A Manual of Library Procedure. (*For the use of Board Members o*

(46) Annual Report (Administrative) of the work of the Central Bo of Irrigation India, 1948-49. (*For the use of Board Members only*).

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